CHAPTER 3
GROUND MOTION AND
GEOLOGICAL HAZARDS ASSESSMENT

3-1. Specification of Ground Motion.

a. General. This document prescribes two ground motions: Ground Motion A and Ground Motion B, as defined in the following paragraphs. The ground motions are expressed in terms of spectral ordinates at 0.20 sec (S_{DS}) and 1.0 sec (S_{D1}). These spectral values are derived from various seismic hazard maps prepared by the U.S. Geological Survey (USGS) and the Building Seismic Safety Council (BSSC) of the National Institute of Building Sciences (NIBS).

b. USGS Seismic Hazard Maps. At the request of the BSSC, USGS prepared probabilistic spectral acceleration maps for ground motions with 10 percent, 5 percent, and 2 percent probability of exceedance in 50 years. For each of these ground motions, probabilistic spectral ordinate maps were developed for peak ground accelerations and spectral response accelerations at 0.2, 0.3, and 1.0 seconds. Additionally, deterministic spectral ordinate maps were developed for areas adjacent to major active faults.

c. Maximum Considered Earthquake (MCE) Maps. In response to concerns regarding the use of the USGS maps by the building design professions, BSSC convened a nation-wide Design Values Group to review the maps and prepare design values for FEMA 302. The concerns of the design profession regarding the probabilistic maps included:

(1) The 10 percent probability of exceedance in 50 years ground motion generally used as a basis of seismic codes did not adequately capture the hazard due to large, but infrequent, events in some areas of the eastern and central U.S.

(2) Probabilistic values near major active faults tended to be very high because of the high rates of activity.

(3) Probabilistic values in some areas that appeared to be unreasonably low could be attributed to lack of sufficient data regarding source zones and frequency of events.

To address these concerns, the Design Values Group developed the MCE maps for spectral ordinates at 0.2 sec (denoted as S_S) and 1.0 sec (denoted as S_1). These maps are generally based on the USGS probabilistic maps for ground motion with 2 percent probability of exceedance in 50 years (approximately 2,500-year return period), but with deterministic values near major active faults and higher threshold values in selected areas of low seismicity. As indicated below, the design spectral ordinates were selected as two-thirds of the site-adjusted MCE values. The traditional seismic risk level considered by most model building codes is 10 percent probability of exceedance in 50 years (return period of about 500 years). Because the value of the ground motion for other risk levels is a function of the shape of the site-specific hazard curve, a valid comparison of the ground motion specified by prior codes with

2/3 of MCE can only be made on a site-specific or regional basis. However, the authors of the FEMA 302 provisions have indicated that, in many areas of the U.S., the new ground motions corresponding to 2/3 of MCE will be comparable to those specified by prior codes. It was also considered that, for most structural elements, the design criteria in FEMA 302 provided adequate reserve capacity to resist collapse at the MCE hazard level.

d. *Site Response Coefficients*. For all structures located within those regions of the maps having values of short-period spectral acceleration, S_S, greater than 0.15g, or values of the one-second period spectral acceleration, S_1, greater than 0.04g, the site shall be classified according to Table 3-1. Based on these Site Classes, FEMA 302 assigns Site Response Coefficients, F_a and F_v, as indicated in Tables 3-2a and 3-2b. The adjusted MCE spectral response acceleration for short periods, S_{MS}, and at 1 second, S_{M1}, are defined as:

$$S_{MS} = F_a S_S \quad (3\text{-}1)$$

$$S_{M1} = F_V S_1 \quad (3\text{-}2)$$

3-2. Design Parameters for Ground Motion A (FEMA 302).

a. *General*. Ground Motion A is the basic design ground motion for the FEMA 302 provisions. The design parameters for Ground Motion A are those used in this document for Performance Objectives 1A (Life Safety) and 2A (Safe Egress for Special Occupancy). The combination of performance levels and ground motions to form performance objectives is described in Paragraphs 4-7, 4-8, and 4-9, and is summarized in Tables 4-3 and 4-4.

b. *Design Spectral Response Accelerations*. The spectral response design values, S_{DS} and S_{D1}, adopted in FEMA 302 are defined as:

$$S_{DS} = 2/3\ S_{MS} \quad (3\text{-}3)$$

$$S_{D1} = 2/3\ S_{M1} \quad (3\text{-}4)$$

For regular structures, 5 stories or less in height, and having a period, T, of 0.5 seconds or less, the spectral accelerations, S_{MS} and S_{M1} need not exceed:

$$S_{MS}\ \#\ 1.5\ F_a \quad (3\text{-}5)$$

$$S_{M1}\ \#\ 0.6\ F_v \quad (3\text{-}6)$$

c. *Seismic Response Coefficients*.

(1) Equivalent Lateral Force (ELF) Procedure. For this procedure the seismic base shear is represented as $V=C_S W$ and the seismic response coefficient, C_S, is determined in accordance with the following equation:

$$C_S = \frac{S_{DS}}{R} \quad (3\text{-}7)$$

where

R = Response modification factor defined in Section 5.2.2 of FEMA 302.

The value of C_S need not exceed the following:

$$C_S = \frac{S_{D1}}{TR} \qquad (3\text{-}8)$$

Table 3-1

Site Classification

Class A	Hard rock with measured shear wave velocity, $\overline{v}_s$ > 5,000 ft/sec (1500 m/s)
Class B	Rock with 2,500 ft/sec < $\overline{v}_s$ ≤ 5,000 ft/sec (760 m/s < $\overline{v}_s$ ≤1500 m/s)
Class C	Very dense soil and soft rock with 1,200 ft/sec < $\overline{v}_s$ ≤ 2,500 ft/sec (360 m/s < $\overline{v}_s$ ≤ 760 m/s) or with either $\overline{N}$ > 50 or $\overline{s}_u$ >2,000 psf (100 kPa)
Class D	Stiff soil with 600 ft/sec ≤ $\overline{v}_s$ ≤ 1,200 ft/sec (180 m/s ≤ $\overline{v}_s$ ≤ 360 m/s) or with either 15≤ $\overline{N}$ ≤ 50 or 1,000 psf ≤ $\overline{s}_u$ ≤ 2,000 psf (50 kPa ≤ $\overline{s}_u$ ≤ 100 kPa)
Class E	A soil profile with $\overline{v}_s$ < 600 ft/sec (180 m/s) or with either $\overline{N}$ < 15, $\overline{s}_u$ < 1,000 psf, or any profile with more than 10 ft (3 m) of soft clay defined as soil with PI >20, *w* ≥ 40 percent, and s_u <500 psf (25 kPa).
Class F	Soils requiring site-specific evaluations: 1. Soil vulnerable to potential failure or collapse under seismic loading such as liquefiable soils; quick and highly sensitive clays; and collapsible, weakly cemented soils. 2. Peats and/or highly organic clays (H > 10 ft [3 m] of peat and/or highly organic clay where H = thickness of soil). 3. Very high plasticity clays (H > 25 ft [8 m] with PI > 75). 4. Very thick soft/medium stiff clays (H > 120 ft [36 m]).

Note: v_s is shear wave velocity; N is Standard Penetration Resistance (ASTM D1586-84), not to exceed 100 blows/ft as directly measured in the field without corrections; s_u is undrained shear strength, not to exceed 5,000 psf (250 kPa) (ASTM D2166-91 or D2850-87). $\overline{v}_s$, $\overline{N}$, and $\overline{s}_u$ are average values for the respective parameters for the top 100 feet of the site profile. Refer to FEMA 302 for the procedure to obtain average values for $\overline{v}_s$, $\overline{N}$, and $\overline{s}_u$.

Exception: When the soil properties are not known in sufficient detail to determine the *Site Class, Site Class D* shall be used. *Site Classes E* or *F* need not be assumed unless the authority having jurisdiction determines that *Site Classes E* or *F* could be present at the site or in the event that *Site Classes E* or *F* are established by geotechnical data.

Table 3-2a
Values of F_a as a Function of a Site Class and Mapped Short-Period Spectral Response Acceleration S_s

	Mapped Spectral Acceleration at Short Periods S_s				
Site Class	**$S_s \leq 0.25$**	**$S_s = 0.50$**	**$S_s = 0.75$**	**$S_s = 1.00$**	**$S_s \geq 1.25$**
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	*
F	*	*	*	*	*

Note: Use straight-line interpolation for intermediate values of S_s.

*Site-specific geotechnical investigation and dynamic site response analyses should be performed.

Table 3-2b
Values of F_v as a Function of a Site Class and Mapped Spectral Response Acceleration at One-Second Period S_1

	Mapped Spectral Acceleration at One-Second Period S_1				
Site Class	**$S_1 \leq 0.10$**	**$S_1 = 0.20$**	**$S_1 = 0.30$**	**$S_1 = 0.40$**	**$S_1 \geq 0.50$**
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	*
F	*	*	*	*	*

Note: Use straight-line interpolation for intermediate values of S_1.

*Site-specific geotechnical investigation and dynamic site response analyses should be performed
.

but shall not be less than:

$$C_S = 0.044\, S_{DS} \qquad (3\text{-}9)$$

where:

T = The fundamental period of the structure. The above equations are shown graphically in Figure 3-1.

(2) Modal Analysis Procedure. The required modal periods, mode shapes, and participation factors shall be calculated by established methods of structural, analysis assuming a fixed-base condition.

(a) General response spectrum. Where a design response spectrum is required in this document, and where site specific procedures are not used, the design response-spectrum curve shall be developed as indicated in Figure 3-2, and as follows:

1. For periods equal or less than T_o, the design spectral response acceleration, S_a, shall be as given by the following equation:

$$S_a = 0.4\, S_{DS} + 0.6\, S_{DS}\, (T/T_o) \qquad (3\text{-}10)$$

Where $T_O = 0.2T_S$ and T_S is defined by Equation 3-13.

2. For periods greater than T_o and less than or equal to T_s, the design spectral response acceleration, S_a, shall be as given by the following equation:

$$S_a = S_{DS} \qquad (3\text{-}11)$$

3. For periods greater than T_s, the design spectral response acceleration shall be as given by the following equation:

$$S_a = \frac{S_{D1}}{T} \qquad (3\text{-}12)$$

where the value of T_s shall be as given by the following equation:

$$T_s = \frac{S_{D1}}{S_{DS}} \qquad (3\text{-}13)$$

(b) Modal base shear. The portion of the base shear contributed by the m^{th} mode, V_m, shall be determined from the following equations:

$$V_m = C_{sm}\overline{W_m} \qquad (3\text{-}14)$$

$$\overline{W_m} = \frac{\left(\sum_{i=1}^{n} w_i f_{im}\right)^2}{\sum_{i=1}^{n} w_i f_{im}^2} \qquad (3\text{-}15)$$

where:

C_{sm} = the modal seismic response coefficient determined below,

$\overline{W_m}$ = the effective modal gravity load including portions of the live load as defined in Sec. 5.3.2 of FEMA 302,

w_i = the portion of the total gravity load of the structure at level i, and

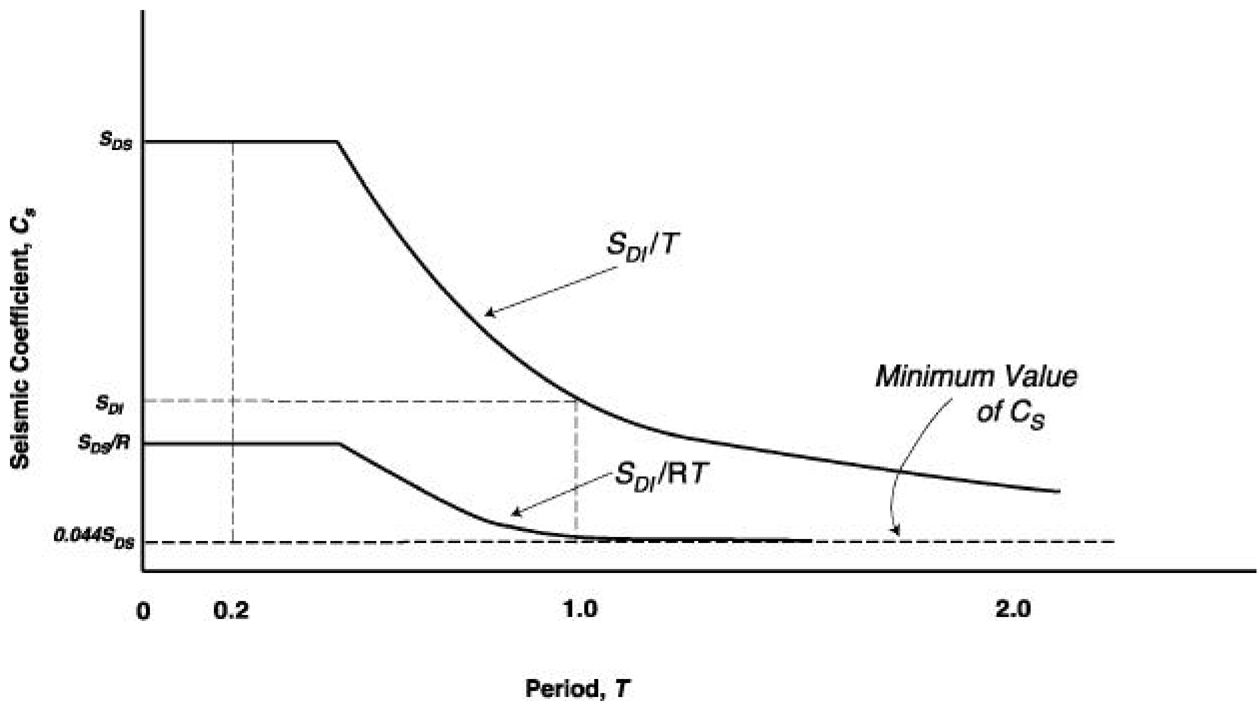


Figure 3-1 Seismic coefficient, C_S.

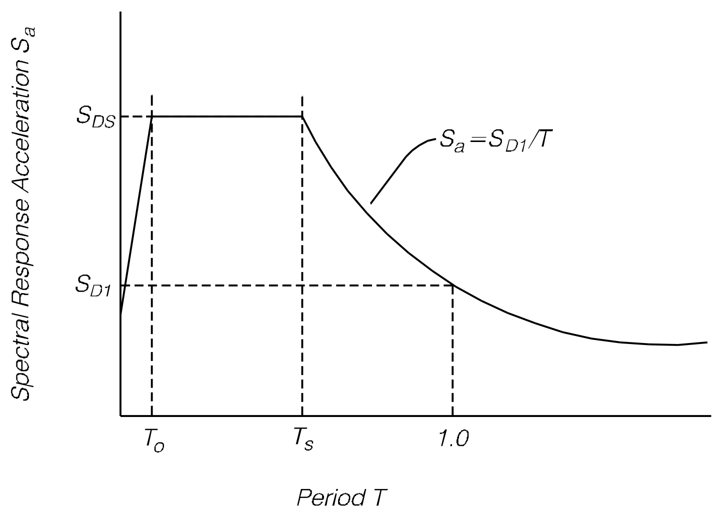


Figure 3-2 Design response spectrum.

f_{im} = the displacement amplitude of the i^{th} level of the structure when vibrating in its m^{th} mode.

The modal seismic response coefficient, C_{sm}, shall be determined in accordance with the following equation:

$$C_{sm} = \frac{S_{am}}{R} \qquad (3\text{-}16)$$

where:

S_{am} = The design response acceleration at period T_m determined from either the general design response spectrum of Paragraph 3-2c (2)(a), or a site-specific response spectrum per Paragraph 3-5,

R = the response modification factor determined from Table 7-1, and

T_m = the modal period of vibration (in seconds) of the m^{th} mode of the structure.

Exceptions:

1. When the general design response spectrum of Paragraph 3-2c (2)(a) is used for structures on Site Class D, E, or F soils, the modal seismic design coefficient, C_{sm}, for modes other than the fundamental mode that have periods less than 0.3 seconds is permitted to be determined by the following equation:

$$C_{sm} = \frac{0.4S_{DS}}{R}(1.0 + 5.0T_m) \qquad (3\text{-}17)$$

Where S_{DS} is as defined in Paragraph 3-2b, and R and T_m, are as defined above.

2. When the general design response spectrum of Paragraph 3-2c(2)(a) is used for structures where any modal period of vibration, T_m, exceeds 4.0 seconds, the modal seismic design coefficient, C_{sm}, for that mode is permitted to be determined by the following equation:

$$C_{sm} = \frac{4S_{D1}}{(R)T_m^2} \qquad (3\text{-}18)$$

Where R, and T_m are as defined above, and S_{D1} is the design spectral response acceleration at a period of 1 second as determined in Paragraph 3-2b.

(c) Modal forces, deflections, and drifts. The modal force, F_{xm}, at each level shall be determined by the following equations:

$$F_{xm} = C_{vxm} V_m \qquad (3\text{-}19)$$

and

$$C_{vxm} = \frac{w_x f_{xm}}{\sum_{i=1}^{n} w_i f_{im}} \qquad (3\text{-}20)$$

where:

C_{vxm} = the vertical distribution factor in the m^{th} mode,

V_m = the total design lateral force or shear at the base in the m^{th} mode,

w_p w_x = the portion of the total gravity load, W, located or assigned to Level i or x.

N_{xm} = the displacement amplitude at the x^{th} level of the structure when vibrating in its m^{th} mode,

and

N_{im} = the displacement amplitude at the i^{th} level of the structure when vibrating in its m^{th} mode.

The modal deflection at each level, $*_{xm}$, shall be determined by the following equations:

$$*_{xm} = C_d \; *_{xem} \qquad (3\text{-}21)$$

and

$$d_{xem} = \left(\frac{g}{4p^2}\right)\left(\frac{T_m^2 F_{xm}}{W_x}\right) \qquad (3\text{-}22)$$

where:

C_d = the deflection amplification factor determined from Table 7-1,

$*_{xem}$ = the deflection of Level x in the m^{th} mode at the center of the mass at Level x determined by an elastic analysis,

g = the acceleration due to gravity (ft/s^2 or m/s^2),

T_m = the modal period of vibration, in seconds, of the m^{th} mode of the structure,

F_{xm} = the portion of the seismic base shear in the m^{th} mode, induced at Level x, and

w_x = the portion of the total gravity load of the structure, W, located or assigned to Level x. The modal drift in a story, $)_m$, shall be computed as the difference of the deflections, $*_{xm}$, at the top and bottom of the story under consideration.

(d) Design values. The design values for the modal base shear, each of the story shear, moment, and drift quantities, and the deflection at each level shall be determined by combining their modal values as obtained above. The combination shall be carried out by taking the square root of the sum of the squares (SRSS) of each of the modal values or by the complete quadratic combinations (CQC) technique.

d. Design values for sites outside the U.S. Table 3-2 in TM 5-809-10 assigns seismic zones to selected locations outside the United States. The seismic zones in that table are consistent with the design values in the 1991 Uniform Building Code (UBC). Table 3-3 in this document provides spectral ordinates that have been derived to provide comparable base shear values.

(1) Algorithms to convert UBC zones to spectral ordinates. The UBC base shear equations are as follows:

$$V = \frac{ZIC}{R_w} W \qquad (3\text{-}23)$$

where

AFRICA:

	S_s	S_I
Algeria:		
Alger	1.24	0.56
Oran	1.24	0.56
Angola:		
Luanda	0.06	0.06
Benin:		
Cotonou	0.06	0.06
Botswana:		
Gaborone	0.06	0.06
Burundi:		
Bujumbura	1.24	0.56
Cameroon:		
Douala	0.06	0.06
Yaounde	0.06	0.06
Cape Verde:		
Praia	0.06	0.06
Central African Republic:		
Bangui	0.06	0.06
Chad:		
Ndjamena	0.06	0.06
Congo:		
Brazzaville	0.06	0.06
Djibouti:		
Djibouti	1.24	0.56
Egypt:		
Alexandria	0.62	0.28
Cairo	0.62	0.28
Port Said	0.62	0.28
Equatorial Guinea:		
Malabo	0.06	0.06
Ethiopia:		
Addis Ababa	1.24	0.56
Asmara	1.24	0.56
Gabon:		
Libreville	0.06	0.06
Gambia:		
Banjul	0.06	0.06
Ghana:		
Accra	1.24	0.56
Guinea:		
Bissau	0.31	0.14
Conakry	0.06	0.06
Ivory Coast:		
Abidijan	0.06	0.06
Kenya:		
Nairobi	0.62	0.28
Lesotho:		
Maseru	0.62	0.28
Liberia:		
Monrovia	0.31	0.14
Libya:		
Tripoli	0.62	0.28
Wheelus AFB	0.62	0.28
Malagasy Republic:		
Tananarive	0.06	0.06
Malawi:		
Blantyre	1.24	0.56
Lilongwe	1.24	0.56
Zomba	1.24	0.56
Mali:		
Bamako	0.06	0.06
Mauritania:		
Nouakchott	0.06	0.06
Mauritius:		
Port Louis	0.06	0.06
Morocco:		
Casablanca	0.62	0.28
Port Lyautey	0.31	0.14
Rabat	0.62	0.28
Tangier	1.24	0.56
Mozambique:		
Maputo	0.62	0.28
Niger:		
Niamey	0.06	0.06
Nigera:		
Ibadan	0.06	0.06
Kaduna	0.06	0.06
Lagos	0.06	0.06
Republic of Rwanda:		
Kigali	1.24	0.56
Senegal:		
Dakar	0.06	0.06
Seychelles		
Victoria	0.06	0.06
Sierra Leone:		
Freetown	0.06	0.06
Somalia:		
Mogadishu	0.06	0.06
South Africa:		
Cape Town	1.24	0.56
Durban	0.62	0.28
Johannesburg	0.62	0.28
Natal	0.31	0.14
Pretoria	0.62	0.28
Swaziland:		
Mbabane	0.62	0.28
Tanzania:		
Dar es Salaam	0.62	0.28
Zanzibar	0.62	0.28
Togo:		
Lome	0.31	0.14
Tunisia:		
Tunis	1.24	0.56
Uganda:		
Kampala	0.62	0.28
Upper Volta:		
Ougadougou	0.06	0.06
Zaire:		
Bukavu	1.24	0.56
Kinshasa	0.06	0.06
Lubumbashi	0.62	0.28
Zambia:		
Lusaka	0.62	0.28
Zimbabwe:		
Harare (Salisbury)	1.24	0.56

ASIA

	S_s	S_I
Afghanistan:		
Kabul	1.65	0.75
Bahrain:		
Manama	0.06	0.06
Bangladesh:		
Dacca	1.24	0.56
Brunei:		
Bandar Seri Begawan	0.31	0.14
Burma:		
Mandalay	1.24	0.56
Rangoon	1.24	0.56
China:		
Canton	0.62	0.28
Chengdu	1.24	0.56
Nanking	0.62	0.28
Peking	1.65	0.75
Shanghai	0.62	0.28
Shengyang	1.65	0.75
Tibwa	1.65	0.75
Tsingtao	1.24	0.56
Wuhan	0.62	0.28
Cyprus:		
Nicosia	1.24	0.56
Hong Kong:		
Hong Kong	0.62	0.28
India:		
Bombay	1.24	0.56
Calcutta	0.62	0.28
Madras	0.31	0.14
New Delhi	1.24	0.56
Indonesia:		
Bandung	1.65	0.75
Jakarta	1.65	0.75
Medan	1.24	0.56
Surabaya	1.65	0.75
Iran:		
Isfahan	1.24	0.56
Shiraz	1.24	0.56
Tabriz	1.65	0.75
Tehran	1.65	0.75

Table 3-3

S_s	S_I	S_s	S_I	S_s	S_I

Table 3-3

	S_s	S_I
Iraq:		
Baghdad	1.24	0.56
Basra	0.31	0.14
Israel:		
Haifa	1.24	0.56
Jerusalem	1.24	0.56
Tel Aviv	1.24	0.56
Japan:		
Fukuoka	1.24	0.56
Itazuke AFB	1.24	0.56
Misawa AFB	1.24	0.56
Naha, Okinawa	1.65	0.75
Osaka/Kobe	1.65	0.75
Sapporo	1.24	0.56
Tokyo	1.65	0.75
Wakkanai	1.24	0.56
Yokohama	1.65	0.75
Yokota	1.65	0.75
Jordan:		
Amman	1.24	0.56
Korea:		
Kwangju	0.31	0.14
Kimhae	0.31	0.14
Pusan	0.31	0.14
Seoul	0.06	0.06
Kuwait:		
Kuwait:	0.31	0.14
Laos:		
Vientiane	0.31	0.14
Lebanon:		
Beirut	1.24	0.56
Malaysia:		
Kuala Lumpur	0.31	0.14
Nepal:		
Kathmandu	1.65	0.75
Oman:		
Muscat	0.62	0.28
Pakistan:		
Islamabad	1.68	0.75
Karachi	1.65	0.75
Lahore	0.62	0.28
Peshawar	1.65	0.75
Quatar:		
Doha	0.06	0.06
Saudi Arabia:		
Al Batin	0.31	0.14
Dhahran	0.31	0.14
Jiddah	0.62	0.28
Khamis Mushayf	0.31	0.14
Riyadh	0.06	0.06
Singapore:		
All	0.31	0.14
South Yemen:		
Aden City	1.24	0.56
Sir Lanka		

	S_s	S_I
Colombo	0.06	0.06
Syria:		
Aleppo	1.24	0.56
Damascus	1.24	0.56
Taiwan:		
All	1.65	0.75
Thailand:		
Bangkok	0.31	0.14
Chinmg Mai	0.62	0.28
Songkhia	0.06	0.06
Udorn	0.31	0.14
Turkey:		
Adana	0.62	0.28
Ankara	0.62	0.28
Istanbul	1.65	0.75
Izmir	1.65	0.75
Karamursel	1.24	0.56
United Arab Emirates:		
Abu Dhabi	0.06	0.06
Dubai	0.06	0.06
Viet Nam:		
Ho Chi Minh City (Saigon)	0.06	0.06
Yemen Arab Republic		
Sanaa	1.24	0.56

ATLANTIC OCEAN AREA

	S_s	S_I
Azorea:		
All	0.62	0.28
Bermuda:		
All	0.31	0.14

CARIBBEAN SEA

	S_s	S_I
Bahama Islands:		
All	0.31	0.14
Cuba:		
All	0.62	0.28
Dominican Republic:		
Santo Domingo	1.24	0.56
French West Indies:		
Martinique	1.24	0.56
Grenada:		
Saint Georges	1.24	0.56
Haiti:		
Port au Prince	1.24	0.56
Jamaica:		
Kingston	1.24	0.56
Leeward Islands:		
All	1.24	0.56
Puerto Rico:		
All	0.83	0.38
Trinidad & Tobago:		
All	1.24	0.56

CENTRAL AMERICA:

	S_s	S_I
Belize:		
Beimopan	0.62	0.28
Canal Zone:		
All	0.62	0.28
Costa Rica:		
San Jose	1.24	0.56
El Salvador:		
San Salvador	1.65	0.75
Guatemala:		
Guatemala	1.65	0.75
Honduras:		
Tegucigalpa	1.24	0.56
Nicaragua:		
Managua	1.65	0.75
Panama:		
Colon	1.24	0.56
Galeta	0.83	0.38
Panama	1.24	0.56
Mexico:		
Ciudad Juarez	0.62	0.28
Guadalajara	1.24	0.56
Hermosillo	1.24	0.56
Matamoros	0.06	0.06
Mazatlan	0.60	0.28
Merida	0.06	0.06
Mexico City	1.24	0.56
Monterrey	0.06	0.06
Nuevo Laredo	0.06	0.06
Tijuana	1.24	0.56

EUROPE

	S_s	S_I
Albania:		
Tirana	1.24	0.56
Austria:		
Salzburg	0.62	0.28
Vienna	0.62	0.28
Belgium:		
Antwerp	0.31	0.14
Brussels	0.62	0.28
Bulgaria:		
Sofia	1.24	0.56
Czechoslovakia:		
Bratislava	0.62	0.28
Prague	0.31	0.14
Denmark:		
Copenhagen	0.31	0.14
Finland:		
Helsinki	0.31	0.14
France:		
Bordeaux	0.62	0.28
Lyon	0.31	0.14
Marseille	1.24	0.56
Nice	1.24	0.56
Strasbourg	0.62	0.28

Table 3-3

	S_s	S_I
Germany, Federal Republic:		
Berlin	0.06	0.06
Bonn	0.62	0.28
Bremen	0.06	0.06
Dusseldorf	0.31	0.14
Frankfurt	0.62	0.28
Hamburg	0.06	0.06
Munich	0.31	0.14
Stuttgart	0.62	0.28
Vaihigen	0.62	0.28
Greece:		
Athens	1.24	0.56
Kavalla	1.65	0.75
Makri	1.65	0.75
Rhodes	1.24	0.56
Sauda Bay	1.65	0.75
Thessaloniki	1.65	0.75
Hungary:		
Budapest	0.62	0.28
Iceland:		
Keflavick	1.24	0.56
Reykjavik	1.65	0.75
Ireland:		
Dublin	0.06	0.06
Italy:		
Aviano AFB	1.24	0.56
Brindisi	0.06	0.06
Florence	1.24	0.56
Genoa	1.24	0.56
Milan	0.62	0.28
Naples	1.24	0.56
Palermo	1.24	0.56
Rome	0.62	0.28
Sicily	1.24	0.56
Trieste	1.24	0.56
Turin	0.62	0.28
Luxembourg:		
Luxembourg	0.31	0.14
Malta:		
Valletta	0.62	0.28
Netherlands:		
All	0.06	0.06
Norway:		
Oslo	0.62	0.28
Poland:		
Krakow	0.62	0.28
Poznan	0.31	0.14
Waraszawa	0.31	0.14
Portugal:		
Lisbon	1.65	0.75
Oporto	1.24	0.56
Romania:		
Bucharest	1.24	0.56
Spain:		
Barcelona	0.62	0.28
Bilbao	0.62	0.28
Madrid	0.06	0.06
Rota	0.62	0.28
Seville	0.62	0.28

	S_s	S_I
Sweden:		
Goteborg	0.62	0.28
Stockholm	0.31	0.14
Switzerland:		
Bern	0.62	0.28
Geneva	0.31	0.14
Zurich	0.62	0.28
United Kingdom:		
Belfast	0.06	0.06
Edinburgh	0.31	0.14
Edzell	0.31	0.14
Glasgow/Renfrew	0.31	0.14
Hamilton	0.31	0.14
Liverpool	0.31	0.14
London	0.62	0.28
Londonderry	0.31	0.14
Thurso	0.31	0.14
U.S.S.R.:		
Kiev	0.06	0.06
Leningrad	0.06	0.06
Moscow	0.06	0.06
Yugoslavia:		
Belgrade	0.62	0.28
Zagreb	1.24	0.56

NORTH AMERICA:

	S_s	S_I
Greenland:		
All	0.31	0.14
Canada:		
Argentia NAS	0.62	0.28
Calgary, Alb	0.31	0.14
Churchill, Man	0.06	0.06
Cold Lake, Alb	0.31	0.14
Edmonton, Alb	0.31	0.14
E. Harmon, AFB	0.62	0.28
Fort Williams, Ont.	0.06	0.06
Frobisher N.W. Ter	0.06	0.06
Goose Airport	0.31	0.14
Halifax	0.31	0.14
Montreal, Quebec	1.24	0.56
Ottawa, Ont	0.62	0.28
St. John's Nfld	1.24	0.56
Toronto, Ont	0.31	0.14
Vancouver	1.24	0.56
Winnepeg, Man	0.31	0.14

SOUTH AMERICA:

	S_s	S_I
Argentina:		
Buenos Aires	0.25	0.10
Brazil:		
Belem	0.06	0.06
Belo Horizonte	0.06	0.06
Brasilia	0.06	0.06
Manaus	0.06	0.06
Porto Allegre	0.06	0.06
Recife	0.06	0.06
Rio de Janeiro	0.06	0.06
Salvador	0.06	0.06
Sao Paulo	0.31	0.14
Bolivia:		
La Paz	1.24	0.56
Santa Cruz	0.31	0.14
Chile:		
Santiago	1.65	0.75
Valparaiso	1.65	0.75
Colombia:		
Bogata	1.24	0.56
Ecuador:		
Quito	1.65	0.75
Guayaquil	1.24	0.56
Paraquay:		
Asuncion	0.06	0.06
Peru:		
Lima	1.65	0.75
Piura	1.65	0.75
Uruguay:		
Montevideo	0.06	0.06
Venezuela:		
Maracaibo	0.62	0.28
Caracas	1.65	0.75

PACIFIC OCEAN AREA:

	S_s	S_I
Australia:		
Brisbane	0.31	0.14
Canberra	0.31	0.14
Melbourne	0.31	0.14
Perth	0.31	0.14
Sydney	0.31	0.14
Caroline Islands:		
Koror, Paulau Is	0.62	0.28
Ponape	0.06	0.06
Fiji:		
Suva	1.24	0.56
Johnson Island:		
All	0.31	0.14
Mariana Islands:		
Guam	1.24	0.56
Saipan	1.24	0.56
Tinian	1.24	0.56
Marshall Islands:		
All	0.31	0.14
New Zealand:		
Auckland	1.24	0.56
Wellington	1.65	0.75
Papau New Guinea:		
Port Moresby	1.24	0.56
Phillipine Islands:		
Cebu	1.65	0.75
Manila	1.65	0.75
Baguio	1.24	0.56
Samoa:		
All	1.24	0.56
Wake Island:		
All	0.06	0.06

Table 3-3

S_s	S_I	S_s	S_I	S_s	S_I

Table 3-3

$$C = \frac{1.25S}{T^{2/3}} \qquad (3\text{-}24)$$

but C need not exceed 2.75.

If the importance factor, I is eliminated in Equation 3-23, and if it is assumed that R_w with allowable stress design is comparable to the FEMA 302 reduction factor, R, with strength design, then by comparison with Equation 3-7,

$$S_{DS} = 2.75Z \qquad (3\text{-}25)$$

Where Z is the seismic zone coefficient from Table 3-4. Similarly, Equation 3-24 can be compared with Equation 3-8 with T = 1.0 sec to yield:

$$S_{D1} = 1.25Z \qquad (3\text{-}26)$$

(2) Spectral ordinates for Seismic Zone O. The Building Seismic Safety Council (BSSC) Design Values Group that developed the MCE maps recommended that, regardless of seismicity, all buildings should be designed to resist a lateral force of one percent of the building weight (i.e. C_z = 0.011x1). If an average value of 4.0 is assumed for the R factor in Equations 3-7 and 3-8, then

$$S_{DS} \text{ and } S_{D1} = 0.04 \qquad (3\text{-}27)$$

(3) Conversion to S_S and S_1. The preceding subparagraph provides the basic relationship between the design parameters in the UBC and those in FEMA 302. It should be noted however, that the Site Adjustment Factor, S, is applied directly to the UBC design values in Equation 3-24, while the FEMA site factors, F_a and F_v, are applied to the MCE ordinates S_S and S_1 in Equations 3-1 and 3-2. The design parameters defined by Equations 3-25 and 3-26 have been multiplied by 1.50 to obtain the equivalent S_S and S_1 values listed in Table 3-3. The adjusted design values, S_{DS} and S_{D1}, for Earthquake A, can thus be obtained by multiplying the values in Table 3-3 by the appropriate local site adjustment factor, F_a or F_v, and multiplying the product by the 2/3 factor indicated in Equations 3-3 and 3-4. Similarly, for Ground Motion B, the product is multiplied by the ¾ factor indicated in Equations 3-28 and 3-29.

(4) Use of available data. As indicated in the above subparagraphs, the spectral ordinates listed in Table 3-3 are derived from the data contained in the current TM 5-809-10. These data are at least six years old, and the conversion is approximate. If better data are available in more recent publications, or from site-specific investigations, the data should be converted to the appropriate design parameters by the procedures outlined in this chapter.

3-3. Design Parameters for Ground Motion B.

a. General. The design parameters for Ground Motion B are those used in this document for Performance Objectives 2B (Safe Egress for Hazardous Occupancy) and 3B (Immediate Occupancy for Essential Facilities). Performance levels, ground motions, and performance objectives are summarized in Tables 4-3, and 4-4. Criteria for the seismic evaluation or design of essential military buildings have typically prescribed ground motion with 5 percent probability of exceedance in 50 years (i.e., a return period of about 1,000 years). This document prescribes three-quarters of the MCE as Ground Motion B for the design of essential buildings. As indicated in Paragraph 3-2, a direct comparison of the ground motion at ¾ of MCE, with that based on 5 percent probability of exceedance in 50 years, can only be made on a site-specific or regional basis. The pragmatic intent of ¾ of MCE was the specification of a ground motion for enhanced performance objectives that would be comparable to that specified in prior military documents.

b. *Design Values*. The design spectral response acceleration parameters, S_{DS} and S_{D1}, for Ground Motion B, shall be in accordance with the following:

$$S_{DS} = \tfrac{3}{4}\, S_{MS} \qquad (3\text{-}28)$$

$$S_{D1} = \tfrac{3}{4}\, S_{M1} \qquad (3\text{-}29)$$

Other design parameters, as defined in Paragraph 3-2 for the ELF or modal analysis procedures, shall be calculated using the above values of S_{DS} and S_{D1} and a response modification factor, R, of 1.0.

3-4. Site-Specific Determination of Ground Motion.

a. *General*. The site-specific determination of ground motion may be used for any structure, and should be considered where any of the following apply:

- The structure is assigned to Performance Objectives 2B or 3B.
- The site of the structure is within 10 kilometers of an active fault.
- The structure is located on Type F soils.
- A time history response analysis of the structure will be performed.
- The structure is to be designed with base isolation or energy dissipation.

Site-specific determination of the ground motion shall be performed only with prior authorization of the cognizant design authority. If a site-specific spectrum is determined for the design ground motion, the spectrum is permitted to be less than the general response spectrum given in Figure 3-2, but not less than 70 percent of that spectrum.

b. *Required Expertise*. Multi-disciplinary expertise is needed for the development of site-specific response spectra. Geological and seismological expertise are required in the characterization of seismic sources. The selection of appropriate attenuation relationships and the conduct of site response analyses requires expertise in geotechnical engineering and strong-motion seismology. Conduct of probabilistic seismic hazard analyses requires expertise in probabilistic modeling and methods. A team approach is therefore often appropriate for site-specific response spectrum development. It is important that the team or lead geotechnical specialist work closely with the design engineer to ensure a common understanding of design earthquakes, approaches to be followed in developing site-specific response spectra, and the nature and limitations of the ground motion outputs developed from the geotechnical studies. The peer review prescribed for Seismic Use Group III buildings in Paragraph 1-9a shall apply to the site-specific determination of ground motion for those buildings.

c. General Approaches. There are two general approaches to developing site-specific response spectra: deterministic approach, and probabilistic approach.

(1) In the deterministic approach, site ground motions are estimated for a specific, selected earthquake; that is, an earthquake of a certain size occurring on a specific seismic source at a certain distance from the site. Often, the earthquake is selected to be the largest earthquake judged to be capable of occurring on the seismic source, or the maximum earthquake, and is assumed to occur on the portion of the seismic source that is closest to the site. After the earthquake magnitude and distance are selected, site ground motions are then deterministically estimated using applicable ground-motion attenuation relationships (see Paragraph 3f below), statistical analyses of ground motion data recorded under similar conditions, or other techniques.

(2) In the probabilistic approach, site ground motions are estimated for selected values of annual frequency or return period for ground motion exceedance, or probability of ground motion exceedance in a certain exposure time (or design time period). The probability of exceeding a certain level of ground motion at a site is a function of the locations of seismic sources and the uncertainty of future earthquake locations on the sources, the frequency of occurrence of earthquakes of different magnitudes on the various sources, and the source-to-site ground motion attenuation, including its uncertainty.

(3) In this document, site specific Ground Motions A and B are determined using both probabilistic and deterministic parameters.

(a) In regions where active faults have not been identified, design ground motions shall be determined using a probabilistic approach as two-thirds (for Ground Motion A) and three-fourths (for ground Motion B) of ground motions having a 2 percent probability of exceedance in 50 years.

(b) In regions where active faults have been identified, ground motions shall be determined using both a probabilistic and a deterministic approach. Design ground motions may be the lesser of: (1) two-thirds (for Ground Motion A) or three-fourths (for Ground Motion B) of ground motions having a probability of exceedance of 2 percent in 50 years; and (2) two-thirds (for Ground Motion A) or three-fourths (for Ground Motion B) of ground motions determined deterministically as one- and one-half times the median (50^{th} percentile) ground motions estimated assuming the occurrence of maximum magnitude earthquakes on portions of active faults closest to the site. Furthermore, in regions having active faults, design ground motions shall not be lower than ground motions that have a 10 percent probability of exceedance in 50 years for Ground Motion A, or 5 percent probability of exceedance in 50 years for Ground Motion B. The following paragraphs provide guidance for conducting a probabilistic ground motion analysis.

d. Overview of Methodology. The development of site-specific response spectra using a probabilistic approach involves the following basic steps: (1) characterizing earthquake sources in terms of their locations and geometrics, maximum earthquake magnitudes, and frequency of earthquake occurrence; (2) characterizing source-to-site ground motion attenuation; (3) carrying out a probabilistic ground motion analysis (often termed a probabilistic seismic hazard analysis, or PSHA) using inputs from (1) and (2); and (4) developing response spectra from the PSHA results. These basic steps are illustrated in Figures 3-3 and 3-4. Figure 3-3 is for the case where a PSHA is carried out for peak ground acceleration (PGA) only, and the response spectrum is then constructed by anchoring a selected response spectrum shape to the value of PGA obtained from the PSHA for the selected probability level. Figure 3-4 is for the case where a PSHA is carried out for response spectral values as well as for PGA, and an equal-probability-of-exceedance (equal-hazard) response spectrum is directly determined from the PSHA results for the selected probability level. The effects of local soil conditions on response spectra are incorporated either directly through the choice of appropriate attenuation relationships or spectral shapes, or by supplemental analyses of site effects in the case where the PSHA is carried out for rock motions at the site. The following paragraphs summarize the different steps involved in developing site-specific response spectra; details of the methodology, including the mathematical formulation of the probabilistic model, are described in Appendix E. Examples of the development of site-specific ground motions using PSHA methodology are also presented in Appendix E. Guidance and computer programs for PSHA are also described in Navy publications TR-2016-SHR and TR-2076-SHR (Ferritto, 1994, 1997).

e. Characterizing Earthquake Sources.

(1) Source identification. Seismic sources are identified on the basis of geological, seismological, and geophysical studies. In the western United States (WUS), i.e., west of the Rocky Mountains) major seismic sources include active faults that have been identified on the basis of surface and subsurface evidence. For example, major active faults in California are shown in map view in Figure 3-5. An example of faults mapped in a localized region of the western U.S. (San Francisco Bay area) is shown in Appendix E, Figure E-10. In some coastal regions of the WUS, specifically northwest California, Oregon, Washington, and southern Alaska, major earthquake sources also include subduction zones, which are regions where a tectonic plate of the earth's crust is thrusting beneath an adjacent tectonic plate. For example, a cross section through the subduction zone in the Puget Sound area of Washington is shown in Figure 3-6. In the eastern U.S. (EUS), earthquake faults typically do not have surface expression, and their subsurface location is usually not precisely known. Accordingly, earthquake sources in the EUS are usually characterized as zones with the zone boundaries selected on the basis of boundaries of geologic structures and/or patterns of seismicity. An example of seismic source zones developed for the EUS is described in Appendix E, Paragraph E-5c.

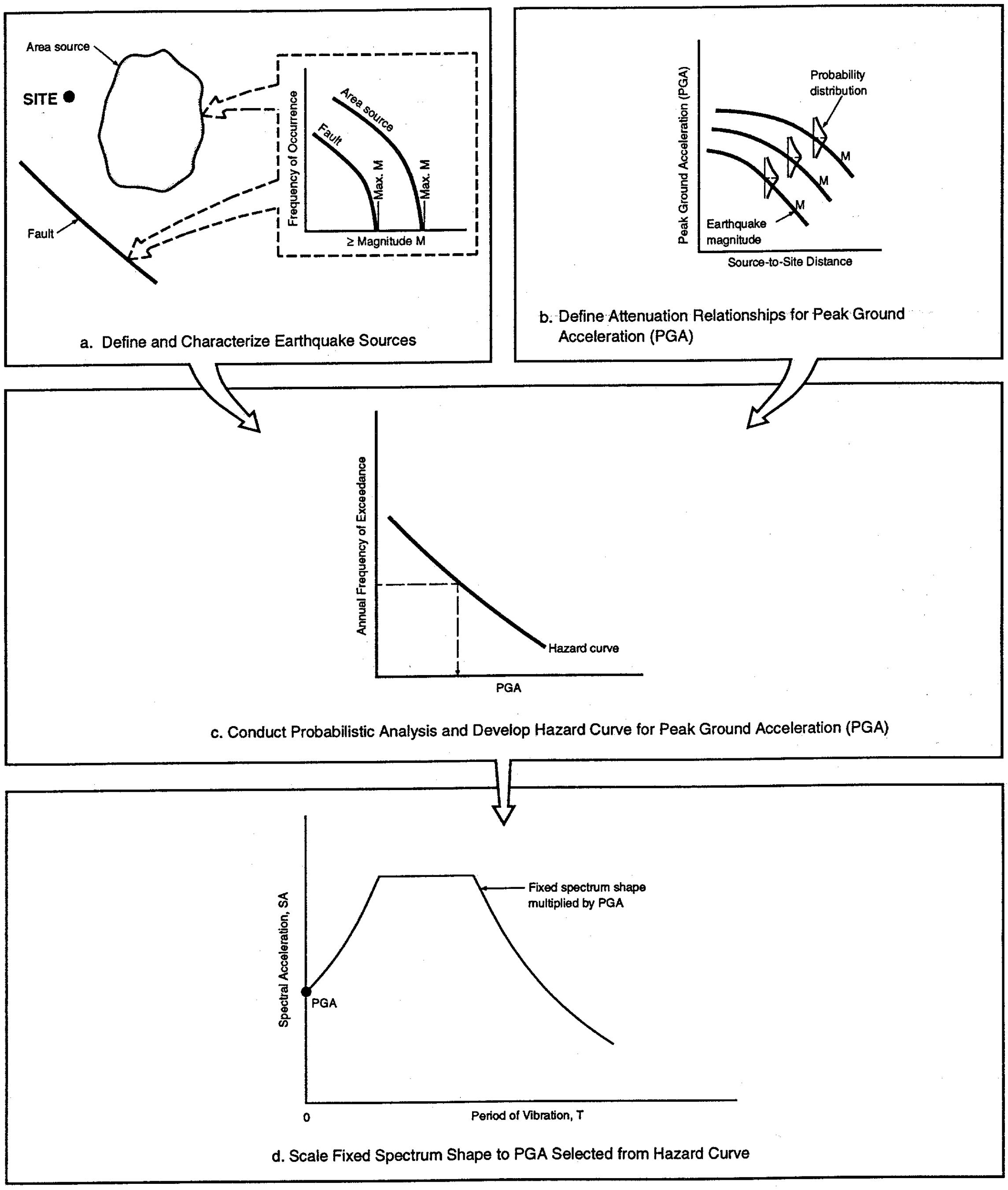


Figure 3-3 **Development of response spectrum based on a fixed spectrum shape and a probabilistic seismic hazard analysis for peak ground acceleration.**

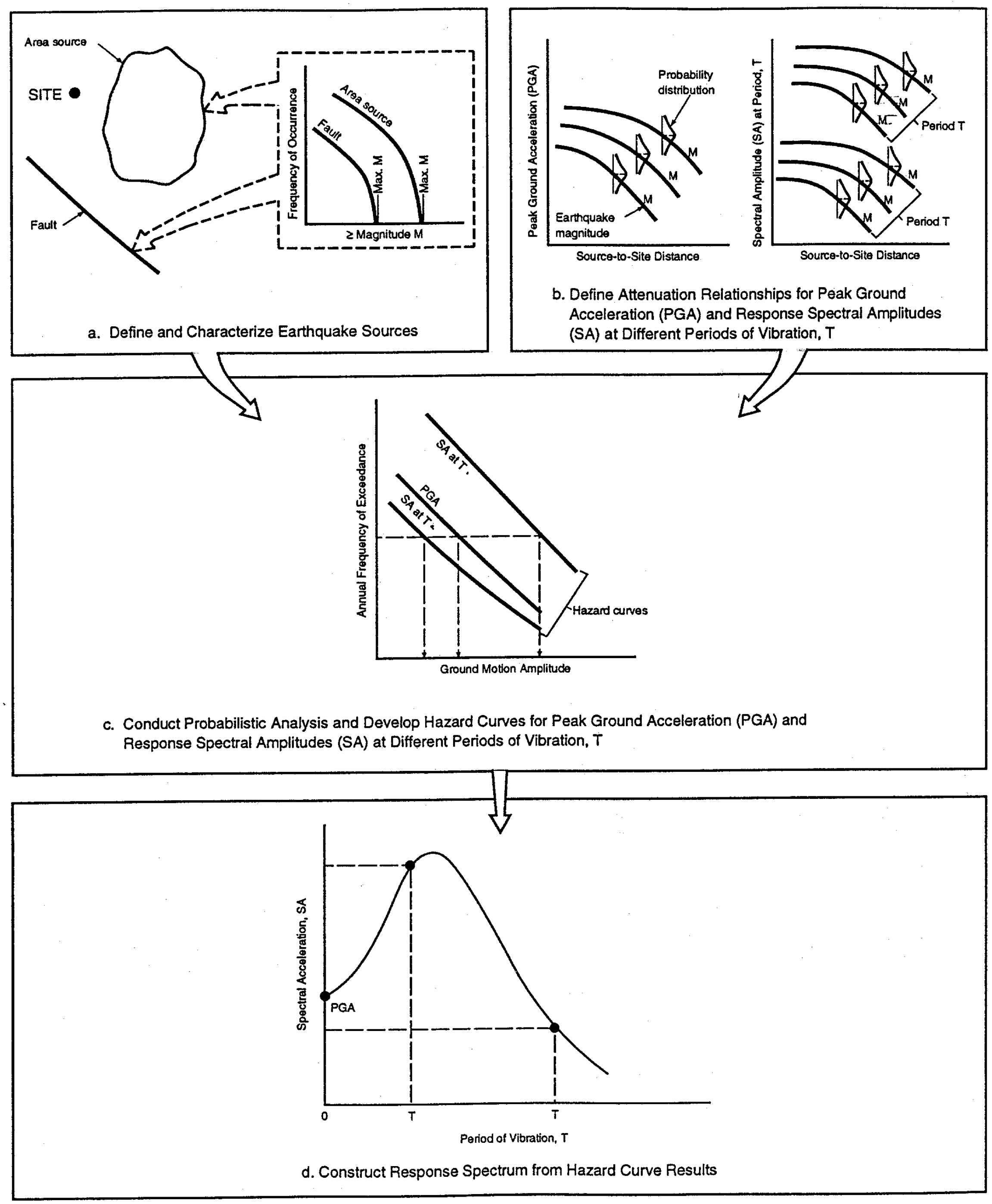


Figure 3-4 **Development of equal-hazard response spectrum from probabilistic seismic hazard analysis for response spectral values.**

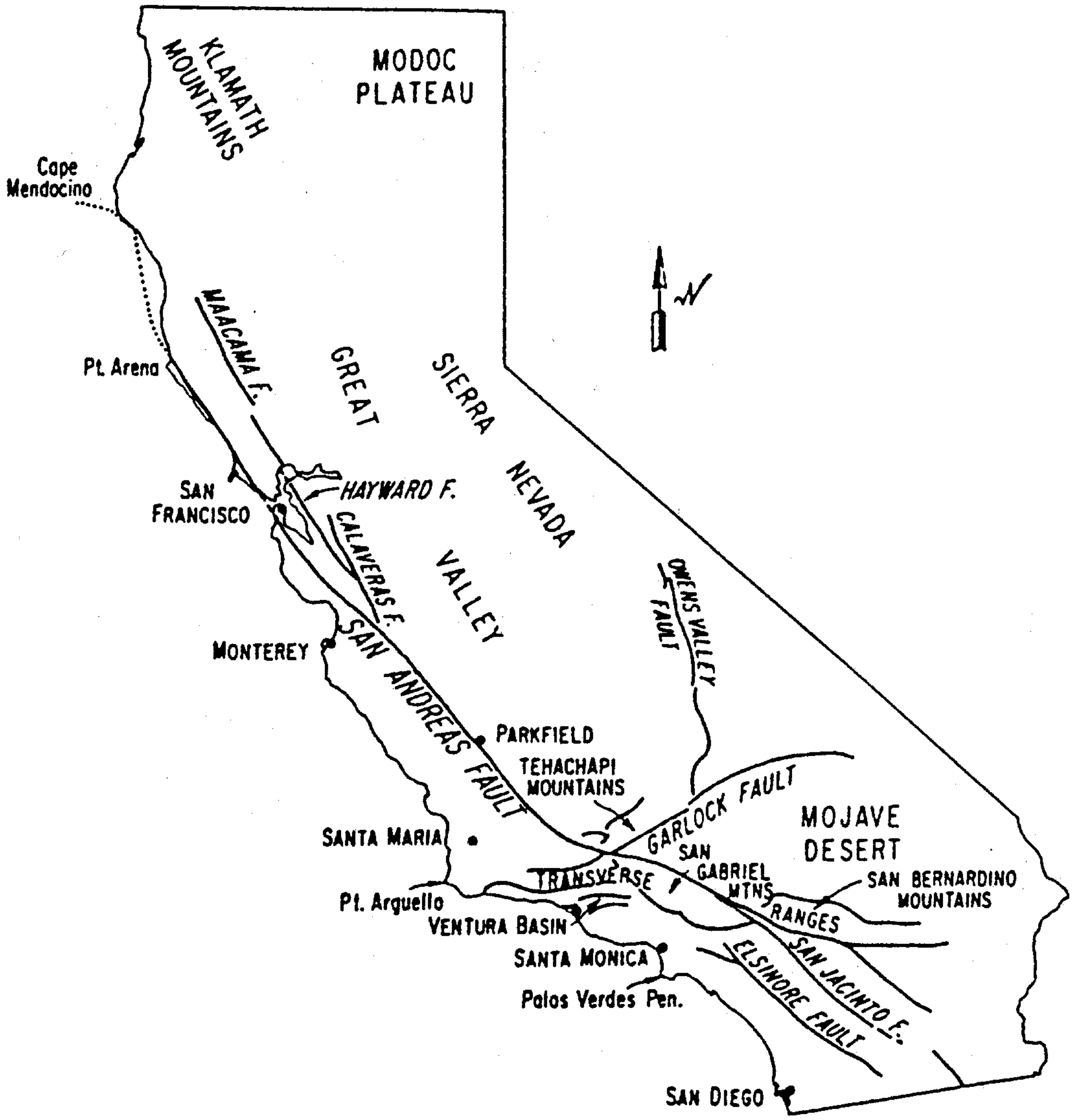
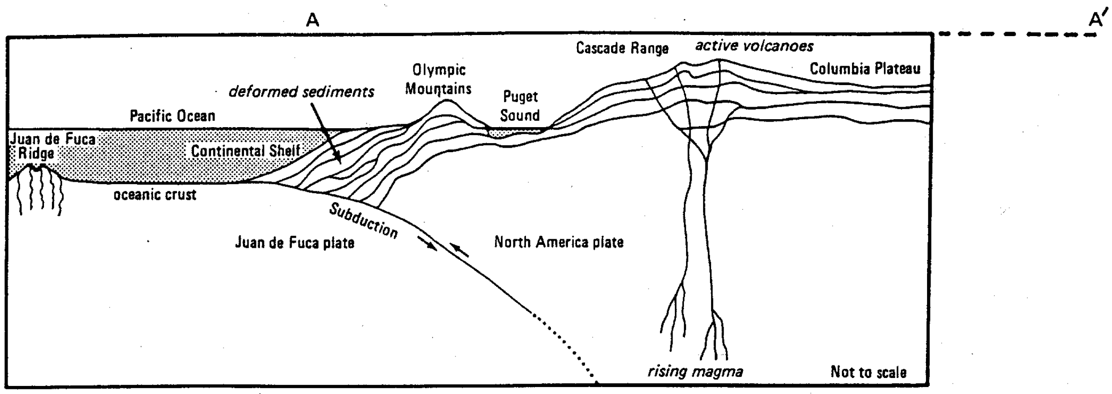
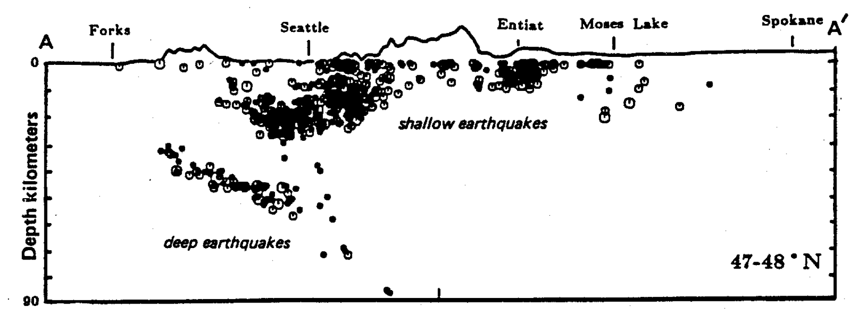


Figure 3-5 **Major active faults in California (after Wesnousky, 1986).**



(a) Geologic and geographic features



(b)) Seismicity (deep earthquakes indicate the slope of the zone of subduction; vertical exaggeration of 2 to 1 for seismicity plot indicates steeper than actual slope of subducting Juan de Fuca plate)

Figure 3-6 Cross section through Puget sound, Washington, showing subduction zone (from Nolson and others, 1988).

(2) Maximum earthquake magnitudes. Maximum magnitude is the physical limit of the size of an earthquake that can be generated by an earthquake source that is related to the dimensions of the source or source segments. For seismic sources in the WUS, maximum magnitudes are usually estimated by assessing the largest dimension (e.g., area) of the source expected to rupture in a single event, and then using empirical relationships that relate earthquake magnitude to rupture size. An example of a correlation between rupture area and earthquake moment magnitude is shown in Figure 3-7. In the EUS, because the source dimensions are typically unknown, there is a greater degree of uncertainty as to the maximum earthquake magnitude. Typically, maximum earthquake magnitudes in the EUS are estimated based on a conservative interpretation of (or extrapolation beyond) the historical seismicity on the source and by analogies to similar geologic structures and tectonic regimes throughout the world. Johnston et al. (1994) present a methodology for assessing maximum earthquake magnitude in the EUS based on an analysis of worldwide data for similar stable continental tectonic regions.

(3) Recurrence relationships. Recurrence relationships characterize the frequency of occurrence of earthquakes of various sizes, from the minimum magnitude of engineering significance to the maximum magnitude estimated for the source. Recurrence relationships are illustrated schematically in diagram A of Figure 3-3 and 3-4.

(a) Earthquake recurrence relationships must be developed for each identified seismic source that could significantly contribute to the seismic hazard at a site. Where earthquake sources are defined as area sources, recurrence relationships are usually developed on the basis of historical seismicity. For sources defined as faults, however, the available historical seismicity for the individual fault is usually insufficient to characterize recurrence rates, particularly for larger earthquakes, and use is typically made of geologic data to supplement the historical records. Geologic data include data on fault slip rates and data from paleo-seismic studies on the occurrence of large prehistoric earthquakes.

(b) Earthquake recurrence curves are usually described by either a truncated exponential recurrence model (Cornell and Vanmarke, 1969) based on Gutenberg and Richter's (1954) recurrence law, or a characteristic earthquake recurrence model (Schwartz and Coppersmith, 1984; Youngs and Coopersmith, 1985a, 1985b). The exponential relationship describes a rate of earthquake occurrence that increases exponentially as earthquake magnitude decreases. On the other hand, the characteristic relationship predicts that a relatively greater number of earthquakes (compared to the exponential relationship) will occur as "characteristic" magnitude events that are at or near the maximum magnitude for the source. A characteristic relationship is illustrated in Figure 3-8. Characteristic and exponential forms of recurrence relationships are compared in Figure 3-9. The exponential relationship is typically used for seismic sources defined as areas, whereas both exponential and characteristic earthquake models are used for individual fault sources. Detailed studies of earthquake recurrence in the Wasatch fault region, Utah, and in the San Francisco Bay region have shown excellent matches between regional seismicity rates and recurrence modeling when combining the

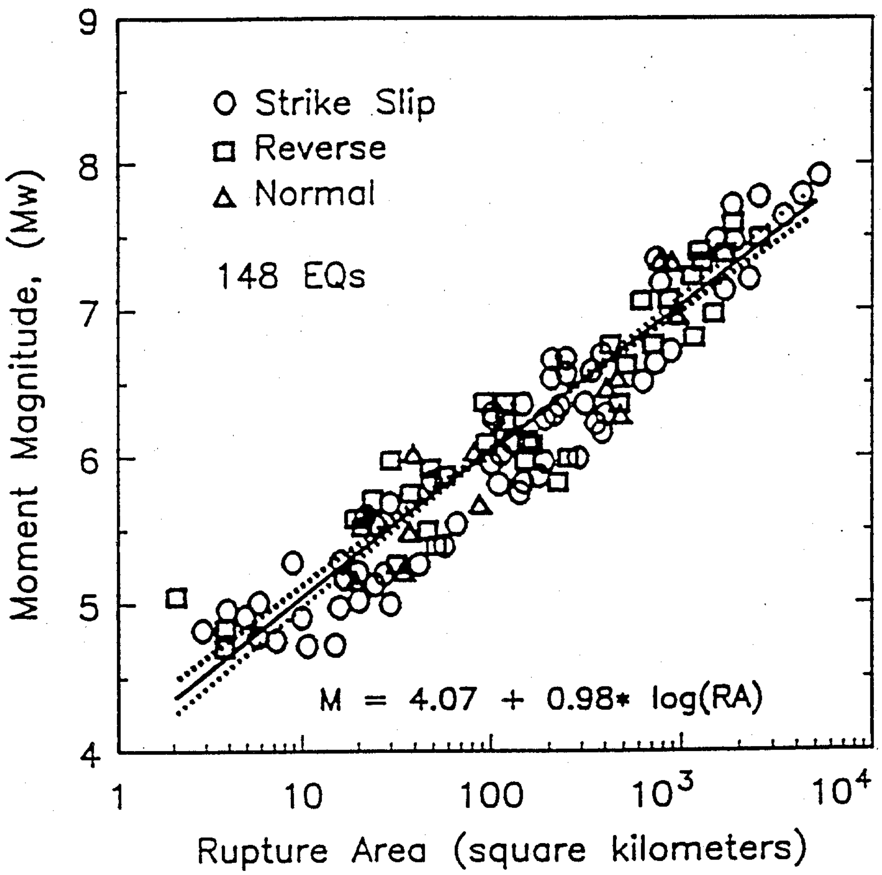


Figure 3-7 Relation between earthquake magnitude and rupture area (after Wells and Coppersmith, 1994).

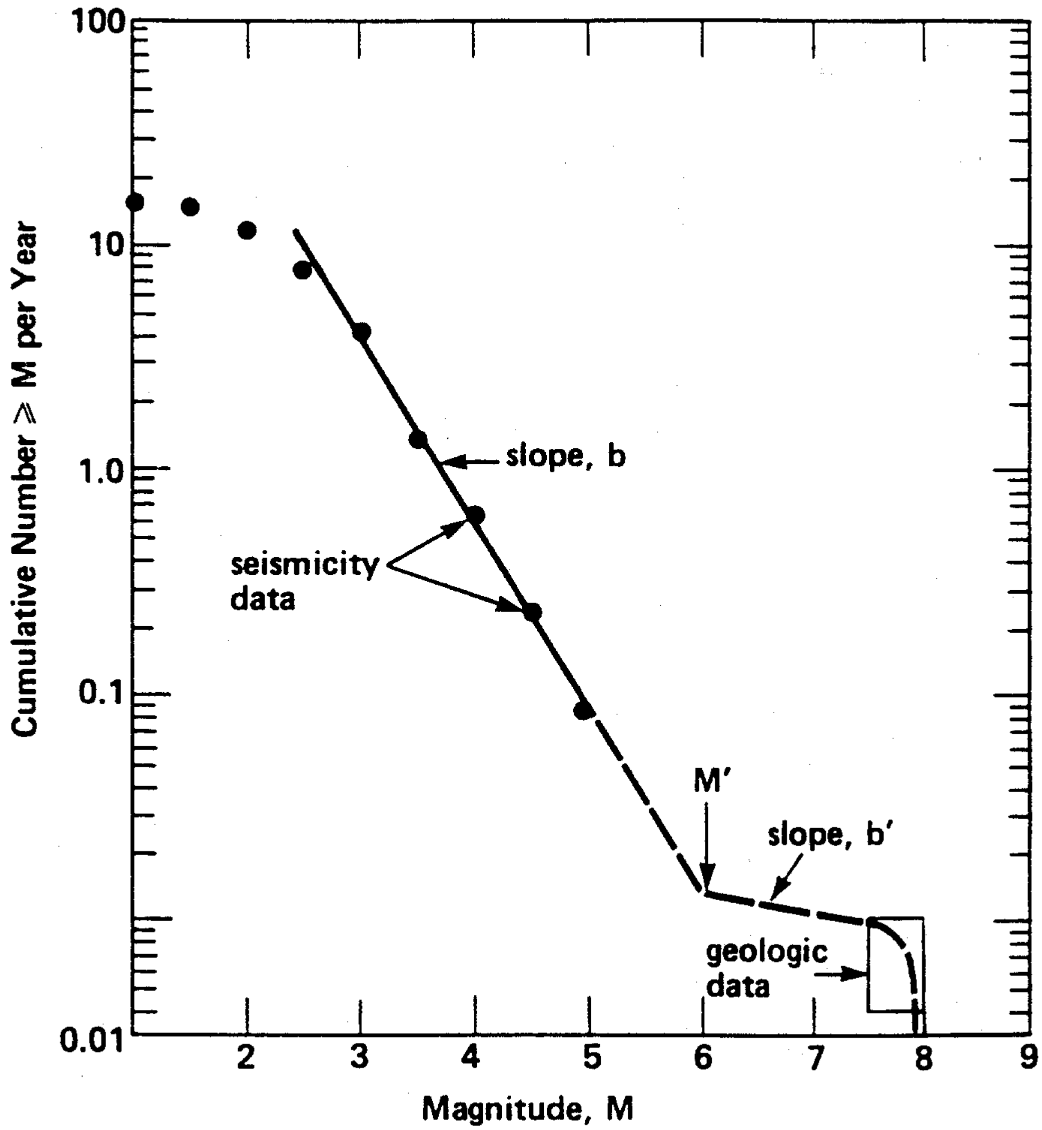


Figure 3-8 **Diagrammatic characteristic earthquake recurrence relationship for an individual fault or fault segment (from Schwartz and Coppersmith, 1984, and National Research council, 1988).**

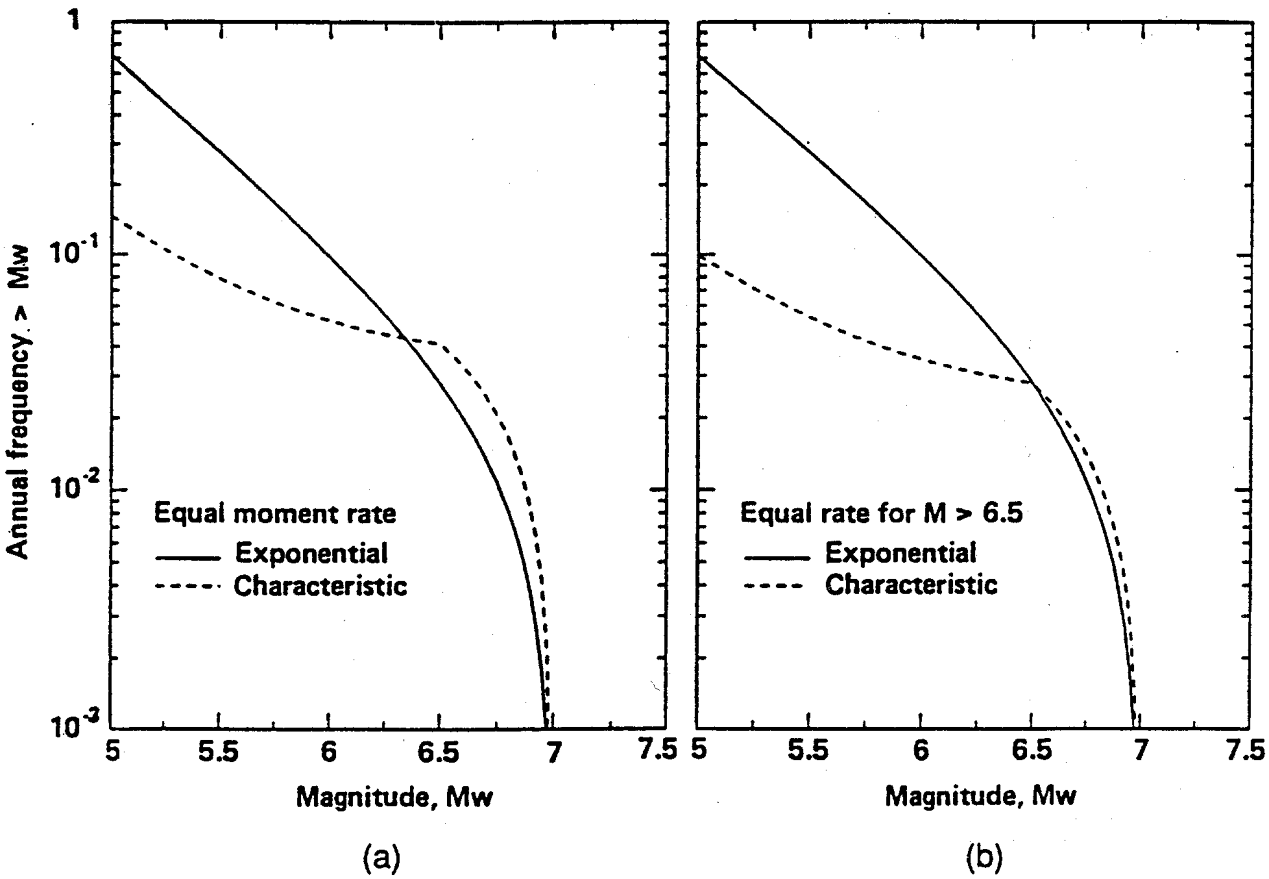


Figure 3-9 **Comparison of exponential and characteristic earthquake magnitude distributions.**

characteristic recurrence model for individual faults with the exponential model for distributed source areas (Youngs et al., 1987, 1988; Youngs et al., 1993); such a comparison is illustrated in the example in Appendix E for the San Francisco Bay region (Paragraph E-5b).

(c) A Poisson probability model is usually assumed for probabilistic ground motion analyses. In the Poisson model, earthquake occurrence in time is assumed to be random and memoryless. The probability of an earthquake in a given time period is thus determined by the average frequency of earthquakes, and is independent of when the last earthquake occurred. This model has been shown to be consistent with earthquake occurrence on a regional basis; however, it does not conform to the process believed to result in earthquakes on an individual fault — one of a gradual accumulation of strain followed by a sudden release. More realistic "real time" earthquake recurrence models have been developed that predict the probability of an earthquake in the next time period, rather than any time period, taking into account the past history (and paleo-history) of large earthquakes on a fault. Usually, there are insufficient geologic and seismic data on the timing of past earthquakes to justify the use of these models; however, real-time recurrence models have been used, for example, in the study of the probabilities of large earthquakes on the San Andreas fault system in Northern California by the Working Group on California Earthquake Probabilities (1990). These models can be considered for site-specific applications when there are sufficient data on the time-dependent occurrence of earthquakes on specific earthquake sources. Further discussion of earthquake recurrence models, including real-time models, is contained in Navy publication TR-2016-SHR (Ferritto, 1994).

f. Characterizing Ground Motion Attenuation.

(1) Attenuation relationships describe the variation of the amplitude of a ground motion parameter as a function of earthquake magnitude and source-to-site distance. A number of attenuation relationships have been developed for PGA and also for response spectral accelerations or velocities for different structural periods of vibration. Figure 3-10 illustrates typical attenuation relationships for PGA and response spectral accelerations for three periods of vibration. These relationships are in terms of earthquake moment magnitude, and the distance is the closest distance to the ruptured fault. The curves in Figure 3-10 are median (50th percentile) relationships. In a probabilistic ground-motion analysis, it is important to include the uncertainty in the ground motion estimates, which reflects the scatter in ground motion data. An example of ground motion data scatter for a single earthquake is illustrated in Figure 3-11. To model this source of uncertainty in ground motion estimation, a probabilistic distribution about the median-curves is assigned, as schematically illustrated in diagram b of Figures 3-3 and 3-4, and as illustrated by the plus-and-minus-one standard deviation curves in Figure 3-11. A log-normal distribution is typically used, and the standard deviation of the distribution is usually provided by the developer of the particular attenuation relationship.

(2) Attenuation relationships have been developed for different tectonic environments, including WUS shallow crustal, EUS, and subduction

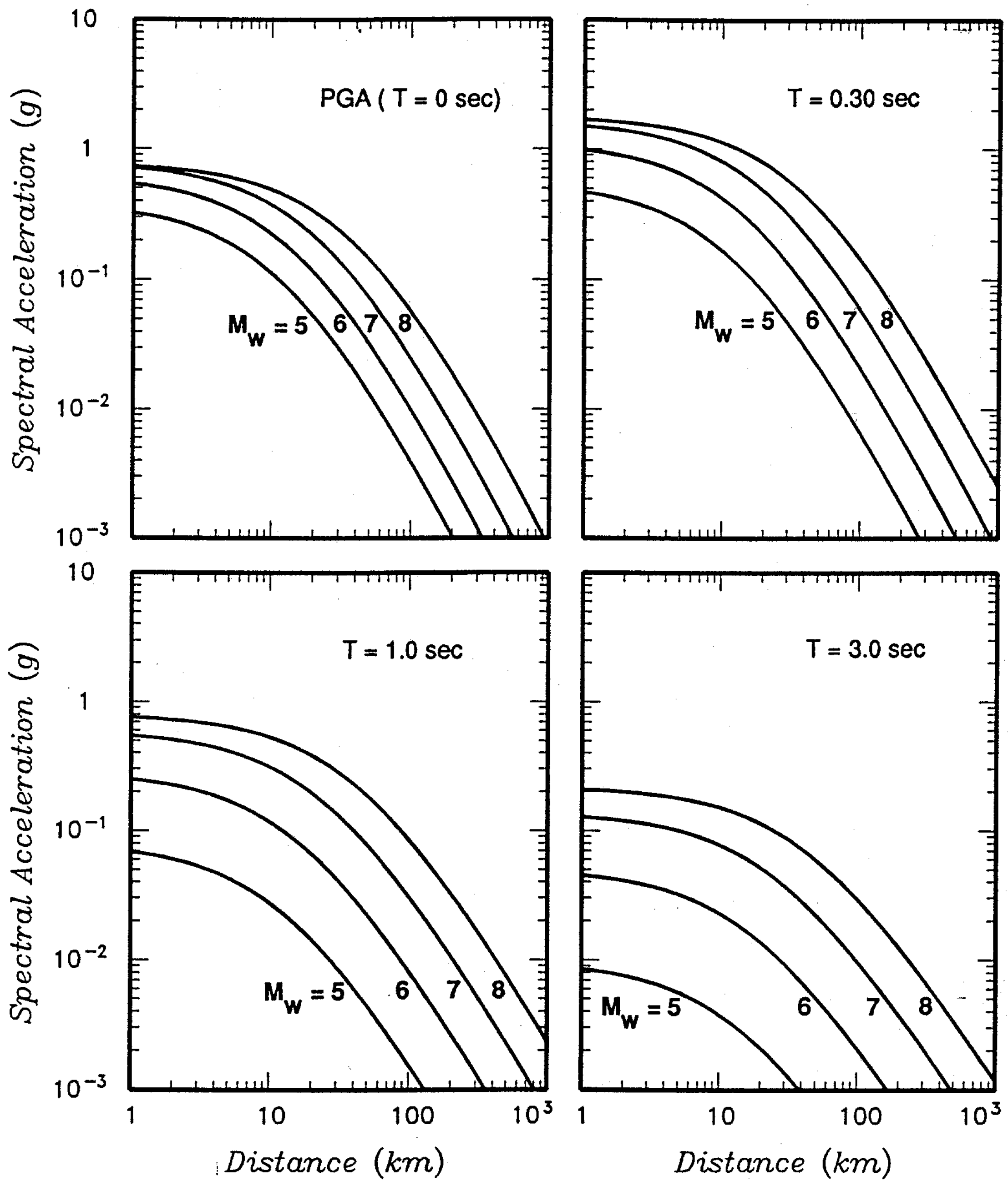


Figure 3-10 **Example of attenuation relationships for response spectral accelerations (5% damping).**

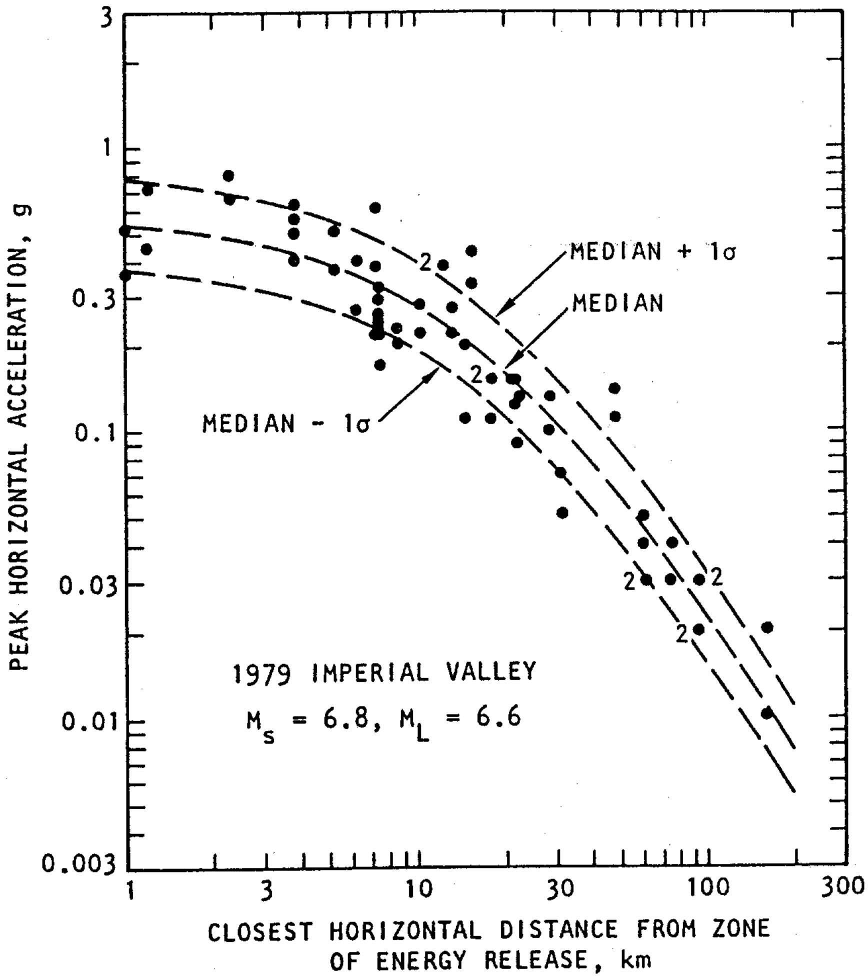


Figure 3-11 **Example of ground motion data scatter for a single earthquake (from Seed and Idriss, 1982).**

zone environments. Attenuation relationships have also been developed for different broad categories of subsurface conditions, particularly for the categories of rock and firm soils. In some cases, attenuation relationships have distinguished the effects of different types of faulting (e.g., strike-slip vs. reverse faulting). It is important to select a set of attenuation relationships that are most applicable to the site under consideration. Several recently developed relationships are summarized in Seismological Research Letters (1997).

g. *Conducting Probabilistic Seismic Hazard Analyses (PSHA).* The seismic source characterization and ground motion attenuation characterization are combined in a probabilistic model to develop relationships between the amplitude of a ground motion parameter and the probability or frequency of its exceedance (diagram c of Figures 3-3 and 3-4). These relationships are termed hazard curves. A hazard curve for PGA is illustrated in Figure 3-12. Appendix E describes the mathematical formulation for the seismic hazard model, and provides examples of its usage in obtaining hazard curves. The appendix also discusses the quantification of uncertainty in hazard curves as related to the uncertainty involved in the relationships and parameters of the model (i.e., uncertainty in seismic source parameters such as maximum earthquake magnitude, frequency of earthquake occurrence, etc., and uncertainty in the choice of appropriate attenuation relationships). It is important to incorporate these uncertainties in a PSHA in order to provide robust estimates of the mean hazard, and evaluate the uncertainties in the hazard.

h. *Developing Response Spectra from the PSHA.* Described below are two alternative approaches for obtaining response spectra based on PSHA: Approach 1 - anchoring a response spectrum shape to the PGAs determined from PSHA; Approach 2 - developing equal-hazard spectra directly from the PSHA. The two approaches are schematically illustrated in Figures 3-3 and 3-4, respectively.

(1) Approach 1 - Anchoring Response Spectrum Shape to PGA Determined from PSHA. In this alternative, the hazard analysis is carried out only for PGA, and the PGAs for the design ground motions are obtained from the hazard curve developed for the site. The response spectra are then constructed by anchoring appropriate response spectrum shapes to the PGA values. Typically, spectrum shapes for the appropriate category of subsurface condition, such as the shapes contained in the 1994 Uniform Building Code (UBC), are used. It should be noted, however, that widely used spectrum shapes, such as those in the 1994 UBC, were developed on the basis of predominantly WUS shallow crustal earthquake ground-motion data, and they may not be appropriate for EUS earthquakes or subduction zone earthquakes. Furthermore, such spectrum shapes are considered to be most applicable to moderate-magnitude earthquakes (magnitude . 6 1/2) and close to moderate distances (distance < 100 km). For larger magnitudes and distances, the shapes may be unconservative in the long-period range; conversely, for smaller magnitudes, the shapes may be overly conservative for long periods. To assess the appropriateness of the spectrum

shapes, the results of a PSHA may be analyzed to determine the dominant magnitude and distance contributions to the seismic hazard. The dominant magnitudes and

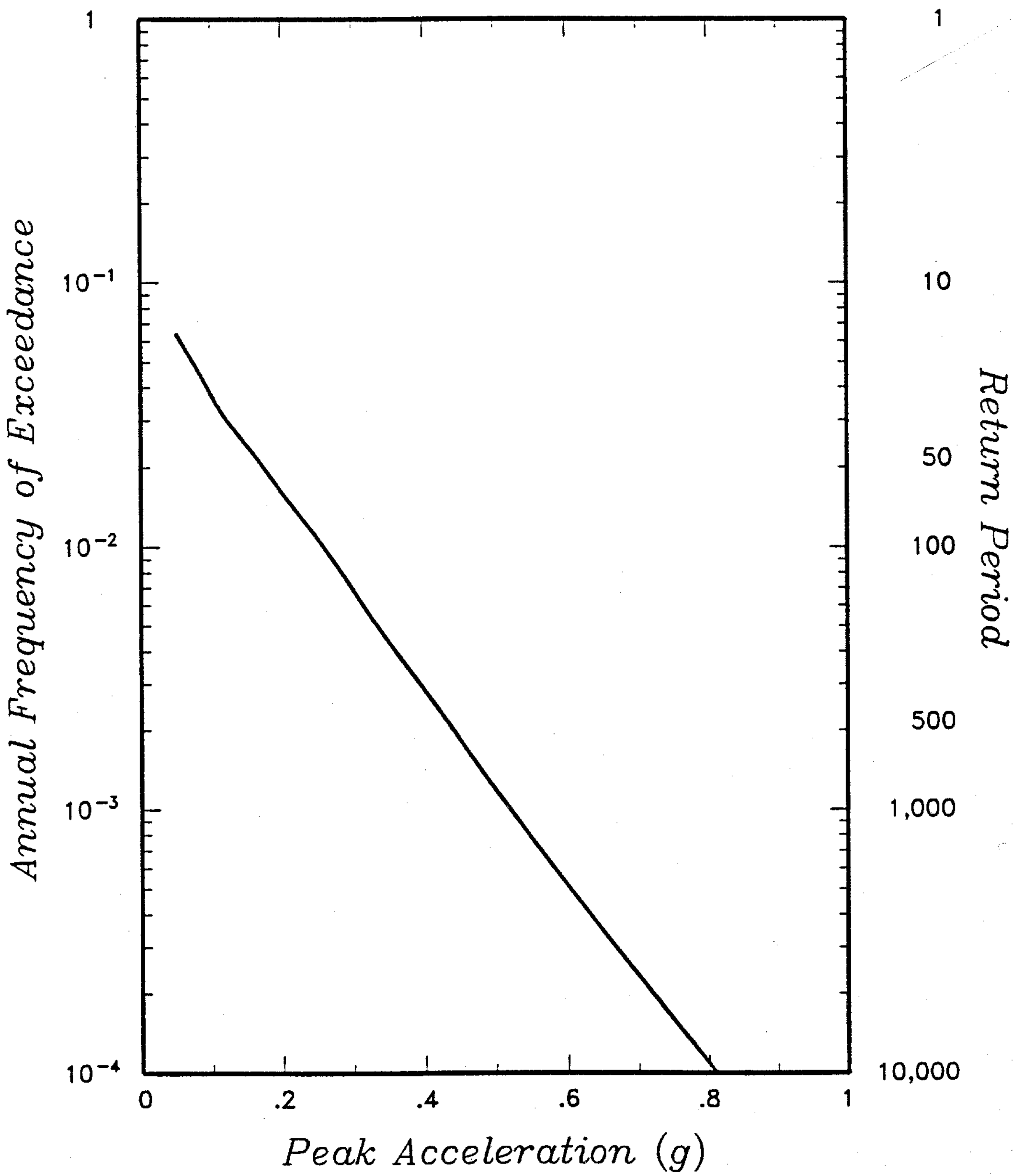


Figure 3-12 **Example seismic hazard curve showing relationship between peak ground acceleration and annual frequency of exceedance.**

distances will, in some cases, differ significantly for different probability levels. Usually the dominant magnitudes increase as the probability of exceedance decreases (e.g., larger dominant magnitudes for Ground Motion B than for Ground Motion A); this is illustrated in Appendix E.

(2) Approach 2 - Developing Equal Hazard Response Spectra Directly From PSHA. In Approach 2, the hazard analysis is carried out for response spectral values at a number of periods of vibration (using response spectral attenuation relationships), as well as for PGA. For the probability levels for the design ground motions, the response spectral values are obtained from the hazard curves, and are then plotted versus period of vibration. A smooth curve is then drawn through the response spectral values obtained for each earthquake, resulting in an equal-hazard response spectrum for each earthquake; that is, a spectrum having the same probability of exceedance at each period of vibration. The process of constructing equal-hazard response spectra from hazard curve results is illustrated in Figure 3-13 for the same site for which the PGA hazard curve was constructed in Figure 3-12. The example in Figure 3-13 is for a return period of 1,000 years, which is approximately equal to the return period for Ground Motion B. (Note in Figure 3-13 that PGA is identically equal to zero-period response spectral acceleration at periods equal to or less than 0.03 second). In general, response spectra should be developed using Approach 2 rather than Approach 1. This is partly because response spectral attenuation relationships needed for Approach 2 are available for both EUS and WUS, and are as reliable as attenuation relationships for PGA. Also, by using Approach 2, the resulting response spectrum will directly incorporate the effects of tectonic environment, magnitude, distance, and probability level on response spectral shape.

i. Accounting for Local Site Effects on Response Spectra.

(1) If the site is a rock site, local soil amplification effects are not applicable, and the response spectrum is directly obtained from the PSHA using attenuation relationships and response spectrum shapes for rock motions.

(2) If the site is a soil site, it is important to account for soil amplification effects on response spectra. Such effects can be very strong in many cases, such as the case illustrated in Figure 3-14, in which ground motions recorded on a soft soil site (Treasure Island) during the 1989 Loma Prieta earth- quake were amplified greatly in comparison to motions recorded on an adjacent rock site (Yerba Buena Island).

(3) Two approaches for incorporating soil amplification effects are: (1) by directly incorporating soil amplification effects in the PSHA through the use of attenuation relationships applicable to the soil conditions at the site; and (2) by developing rock response spectra at the site from a PSHA using rock attenuation relationships, and then carrying out site response analyses to assess the modifying influence of the soil column on the ground motions. The choice between Approaches 1 and 2 depends on whether attenuation relationships are available that are sufficiently applicable to the

soil conditions at the site (Approach 1), and whether site soil conditions are known in sufficient detail to be modeled for site

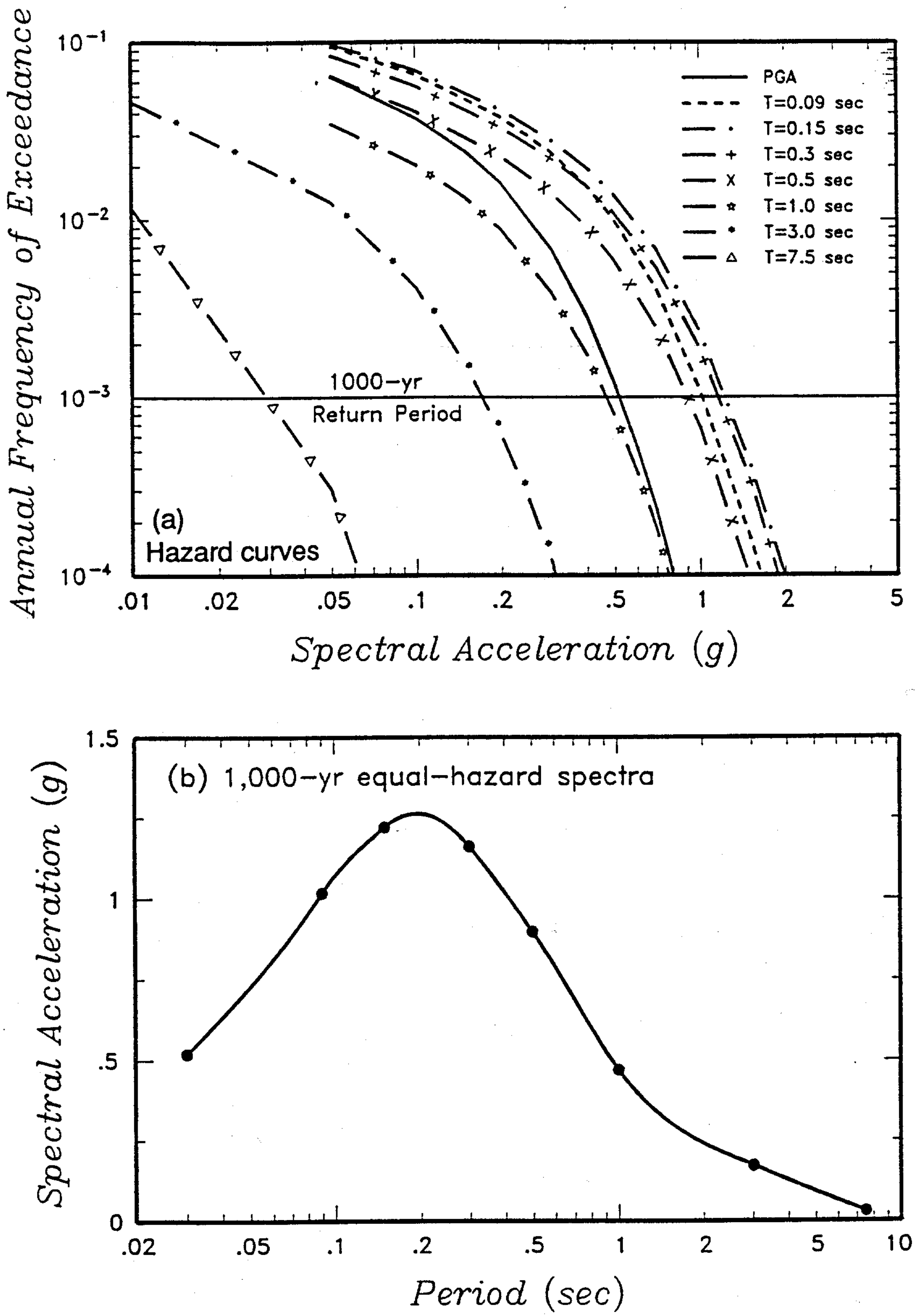


Figure 3-13 Construction of equal-hazard spectra.

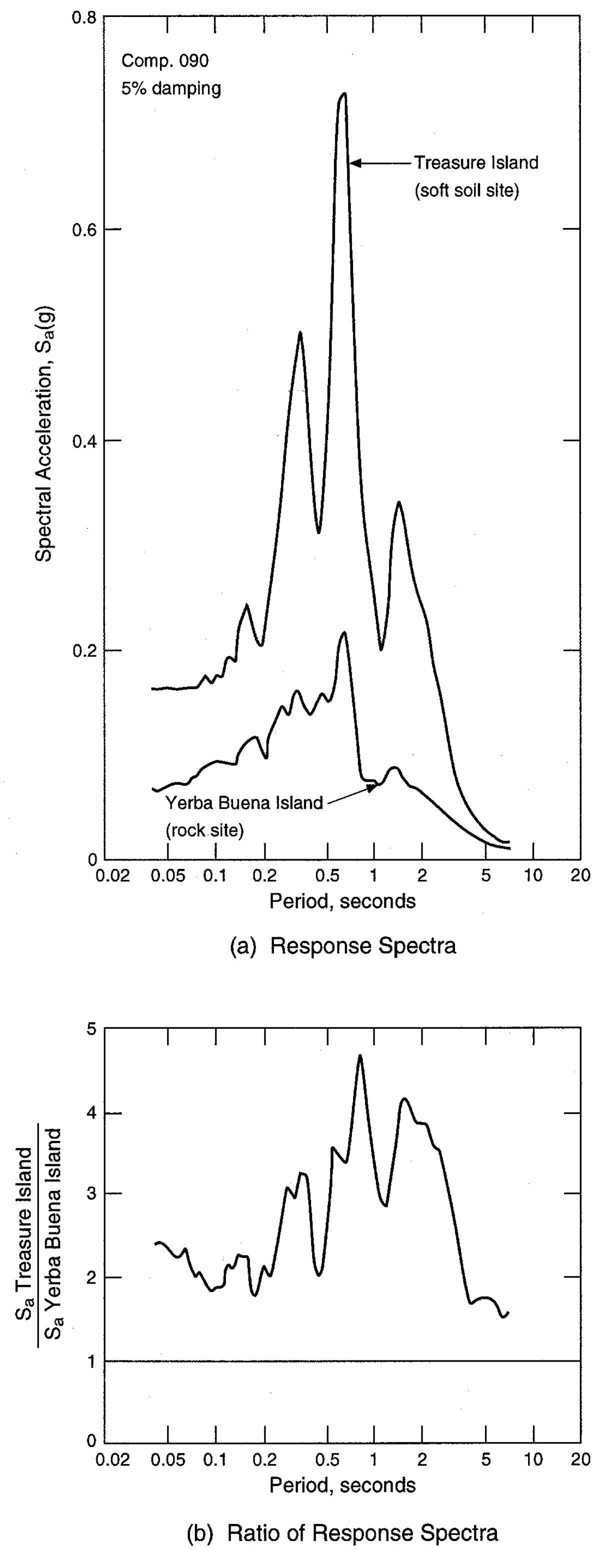


Figure 3-14 **Response spectra and ratio of response spectra for ground motions recorded at a soft site and nearby rock site during the 1989 Loma Prieta earthquake.**

response analysis (Approach 2). Approach 2 can always be considered as an alternative or supplement to approach 1.

(4) Soil amplification effects are stronger for soft clay soils than for stiff clays or dense sands, especially in the long-period range. Soil amplification is also increased by a large change in stiffness or shear wave velocity between the soils and underlying bedrock; therefore, it is particularly appropriate to conduct site response analyses when these conditions are present at a site.

(5) Site response analysis methodology is schematically illustrated in Figure 3-15. The soil profile between the ground surface and underlying rock is modeled in terms of its stratigraphy and dynamic soil properties. Acceleration time histories that are representative of the estimated rock motions are selected, and are propagated through the modeled soil profile using nonlinear or equivalent linear response analytical methods, and top-of-soil motions are obtained. As in other types of theoretical modeling and numerical analyses, site response analyses are sensitive to the details of the analytical procedures, soil dynamic properties, and input motions. The sensitivities should be carefully examined when these analyses are conducted.

(6) In certain cases, it may be appropriate to consider other types of site effects in developing site-specific ground motions. These include surface topographic effects when the surface topography is very irregular and could amplify ground motions, and subsurface basin or buried valley response effects when such two- and three-dimensional effects could significantly modify ground motions in comparison to the one-dimensional site response effects that are usually modeled.

j. Special Characteristics of Ground Motion for Near-Source Earthquakes. At close distances to the earthquake source, within approximately 10 to 15 km of the source, earthquake ground motions often contain a high energy pulse of medium- to long-period ground motion (at periods in the range of approximately 0.5 second to 5 seconds) that occurs when fault rupture propagates toward a site. It has also been found that these pulses exhibit a strong directionality, with the component of motion perpendicular (normal) to the strike of the fault being larger than the component parallel to the strike (see, for example, Somerville et al., 1997). These characteristics of near-source ground motions are illustrated in Figure 3-16, which shows the acceleration, velocity, and displacement time histories and response spectra of the Rinaldi recording obtained during the 1994 Northridge earthquake. These ground-motion characteristics should be incorporated in developing design response spectra, and when required, acceleration time histories for near-source earthquakes.

k. Vertical Ground Motions. For the design of some structures, it may be necessary to analyze the structure for vertical, as well as horizontal, ground motions. Generally, vertical design response spectra are obtained by applying vertical-to-horizontal ratios to horizontal design response spectra. Recent studies (e.g., Silva, 1997) indicate that vertical-to-horizontal response spectral ratios are a function of period of vibration, earthquake source-to-site distance,

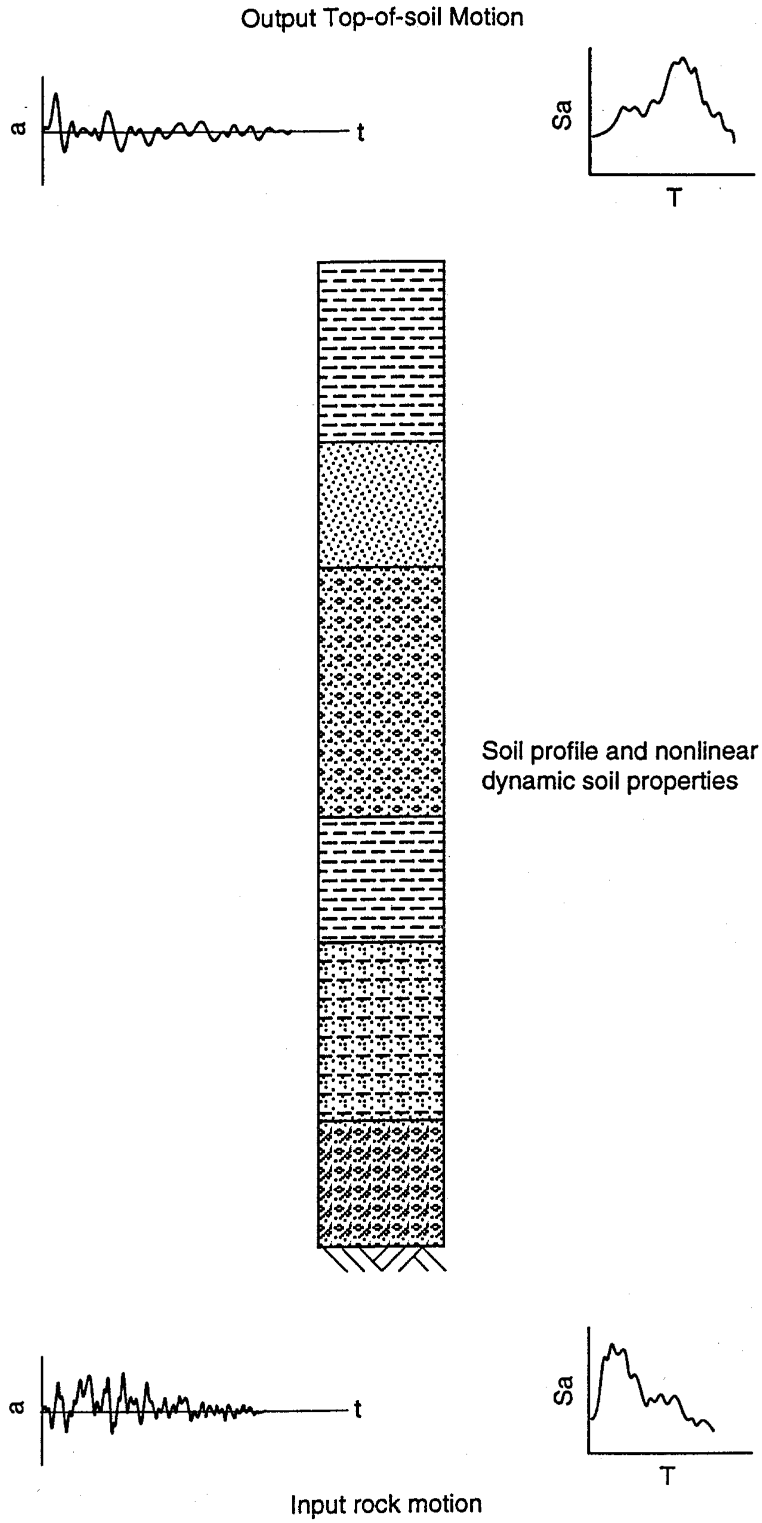
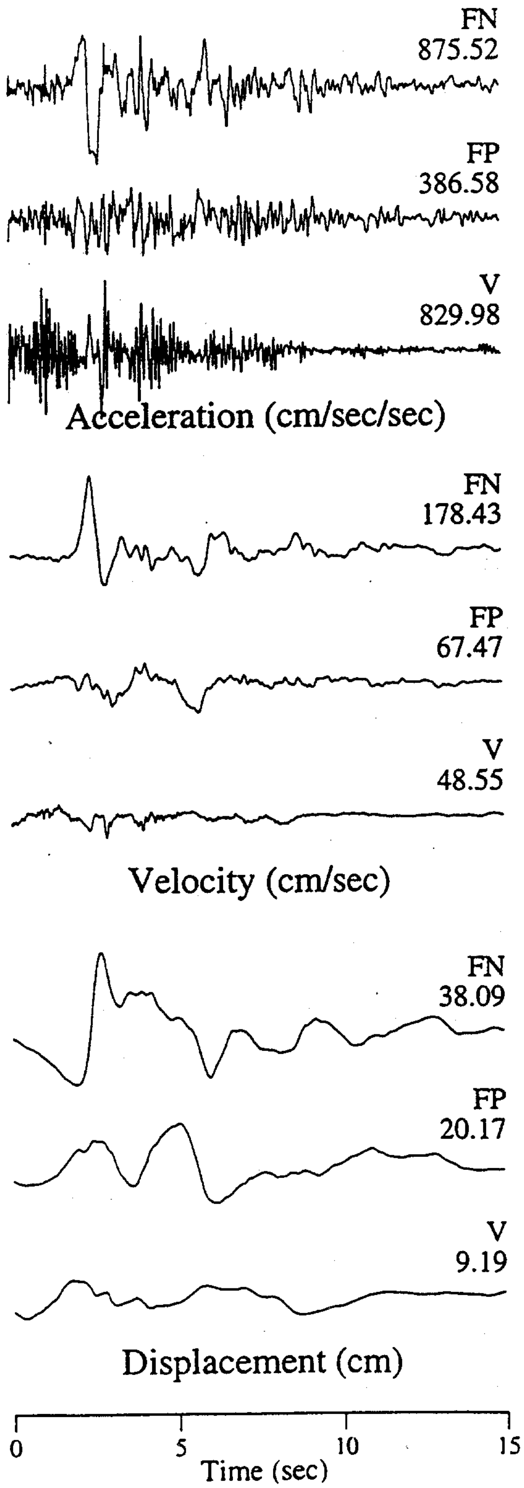


Figure 3-15 Schematic of site response analysis.



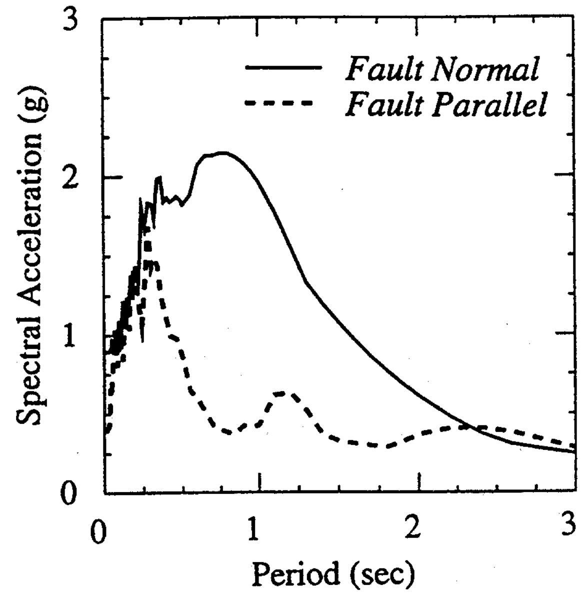


Figure 3-16 **Acceleration and velocity time histories for the strike-normal and strike-parallel horizontal components of ground motion, and their 5% damped response spectra, recorded at Rinaldi during the 1994 Northridge earthquake (Somerville, 1997).**

earthquake magnitude, tectonic environment (W.U.S. and E.U.S.) and subsurface conditions (soil and rock).

Figure 3-17 illustrates trends for these ratios as a function of period of vibration, source-to-site distance, and subsurface conditions for shallow crustal W.U.S. earthquakes of moment magnitude approximately equal to 6.5. The figure illustrates that the commonly used vertical-to-horizontal spectral ratio of two-thirds is generally conservative for longer-period ground motions, but is generally unconservative for short-period ground motions from near-source earthquakes. In fact, these ratios may significantly exceed 1.0 in some cases, as shown in Figure 3-17. Ratios such as those presented in Figure 3-17 may be used to construct design vertical response spectra of ground motions. However, the longer period (greater than 0.2 second) spectral values should be carefully examined, and it may be desirable to adopt for design vertical-to-horizontal spectral ratios for longer periods that are higher than the ratios shown in Figure 3-17.

3-5. Geologic Hazards.

Although, the hazard of strong ground shaking is generally the principal cause of damage to buildings and other structures during earthquakes, other seismic-geologic hazards have caused catastrophic damage to structures during earthquakes. These hazards include:

- *Surface fault rupture*, which is the direct, shearing displacement occurring along the surface trace of the fault that slips during an earthquake.

- *Soil liquefaction*, in which certain types of soil deposits below the groundwater table may lose a substantial amount of strength due to strong earthquake ground shaking, potentially resulting in reduced foundation-bearing capacity, lateral spreading, settlement, and other adverse effects.

- *Soil differential compaction*, which refers to the densification of soils and resulting settlements that may occur due to strong ground shaking.

- *Landsliding* of soil and rock masses on hillside slopes, due to earthquake-ground-shaking-induced inertia forces in the slope.

- *Flooding* induced by earthquakes, which includes the phenomena of tsunami, seiche, and dam, levee, and water tank failures.

The sites of all new buildings shall be evaluated to minimize the possibility that a structure which is adequately resistant to ground shaking could fail due to the presence of a severe site geologic hazard. Guidelines for conducting a geologic hazards study are described in Appendix F. As described in Appendix F, a screening procedure may be applied initially to ascertain whether the possibility of one or more geologic hazards can be screened out for a facility site. For those hazards that cannot be screened out, more detailed procedures should be used to evaluate whether a significant hazard exists, and if necessary, to develop hazard mitigation measures. Guidelines for more detailed evaluations of hazards and for hazard mitigation are also presented in Appendix F. Examples of geologic hazards studies are provided in Appendix G.

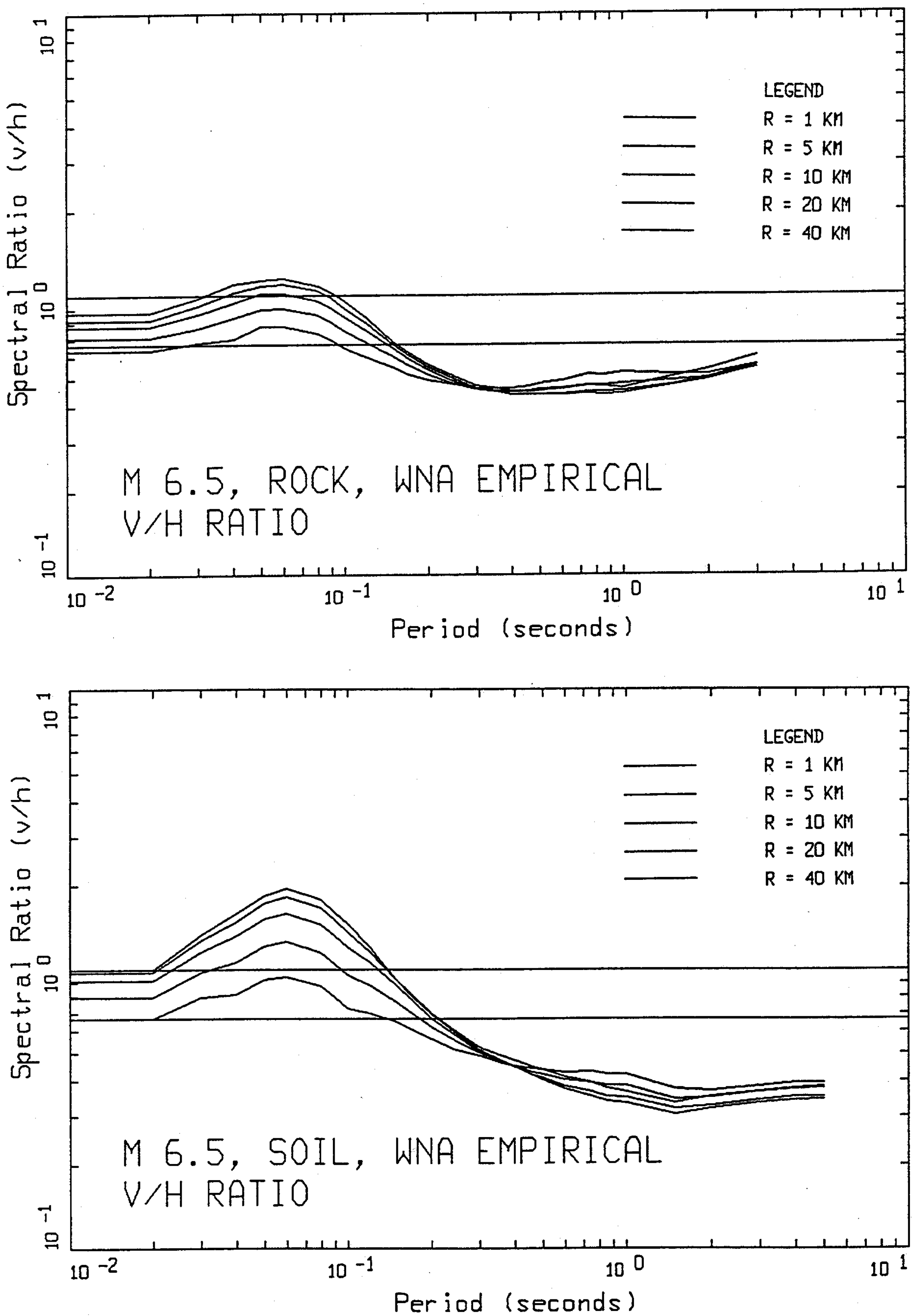


Figure 3-17 Distance dependency of response spectral ratio (V/H) for M 6.5 at rock and soil sites in western North America. Line at 0.66 indicates the constant ratio of 2/3 (Silva, 1997).